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TRANSACTIONS.

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744.

Vol. XXXIII.—March, 1895.

THE BRIDGE OVER THE TENNESSEE RIVER AT JOHNSONVILLE, TENNESSEE.

By HUNTER MCDONALD, M. Am. Soc. C. E.

READ JANUARY 2D, 1895.

WITH DISCUSSION.

The preparation of a more elaborate paper on this subject was nearly complete when paper No. 713* was presented to the Society. It became necessary, therefore, to revise the original and strike out, as far as possible, those features of the work already described.

The bridge of the Nashville, Chattanooga and St. Louis Railway over the Tennessee River, at Johnsonville, Tenn., was built in 1866 by the Nashville and Northwestern Railroad Company. Brig.-Gen. William P. Innes was Chief Engineer, L. Hutchinson, Resident Engineer, and Charles H. Ball, Assistant Engineer. D. L. McMillen & Co. were the chief contractors, and Flannery & Nash, subcontractors.

* *Transactions*, Vol. XXXI, page 587.

The cost of the substructure was $94 512, and of the superstructure, $73 132. No plans or records (save the minutes of the Board of Directors) are now extant to show the details of the work.

The first bridge was a Howe truss, which was replaced by a combination bridge in 1871, designed for a moving load of 2 240 lbs. per lineal foot.

Mr. Charles H. Ball, who is still living, in a letter dated at Waverly, Tenn., gives the following description of the preparation of the foundations:

"The average depth of the water was between 6 and 7 ft., except at the channel on the west side of the draw, where it was 10 ft. I examined the river-bed, and it was composed of hard blue clay cemented with iron pyrites so hard it was difficult to drive a steel bar into it 6 ins. It was very smooth and nearly level up and down the river the length of the piers.

"The piers were built upon cribs of timber, sweet gum, cypress, oak and hickory, roughly hewn. These cribs were built in alternate layers, with sticks at right angles and bolted together. They were towed out to place between two barges, the stone foundation laid roughly on them and grouted. When the crib rested on the bottom, the masonry was far enough above the water to allow stone to be set. The cribs were perhaps 12 ins. larger than the masonry all around.

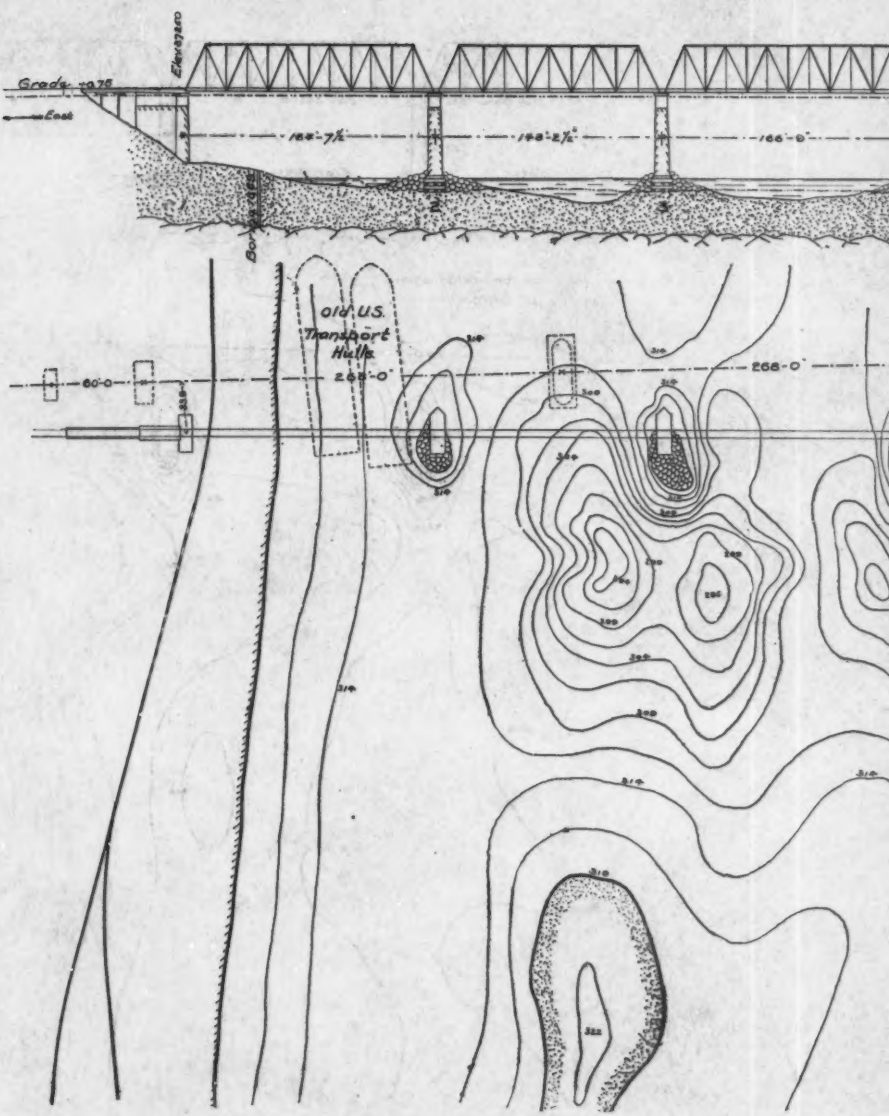
"In my report on the subject of the foundation of the bridge I protested against building the piers without going down to solid rock and disclaimed responsibility for undermining by water.

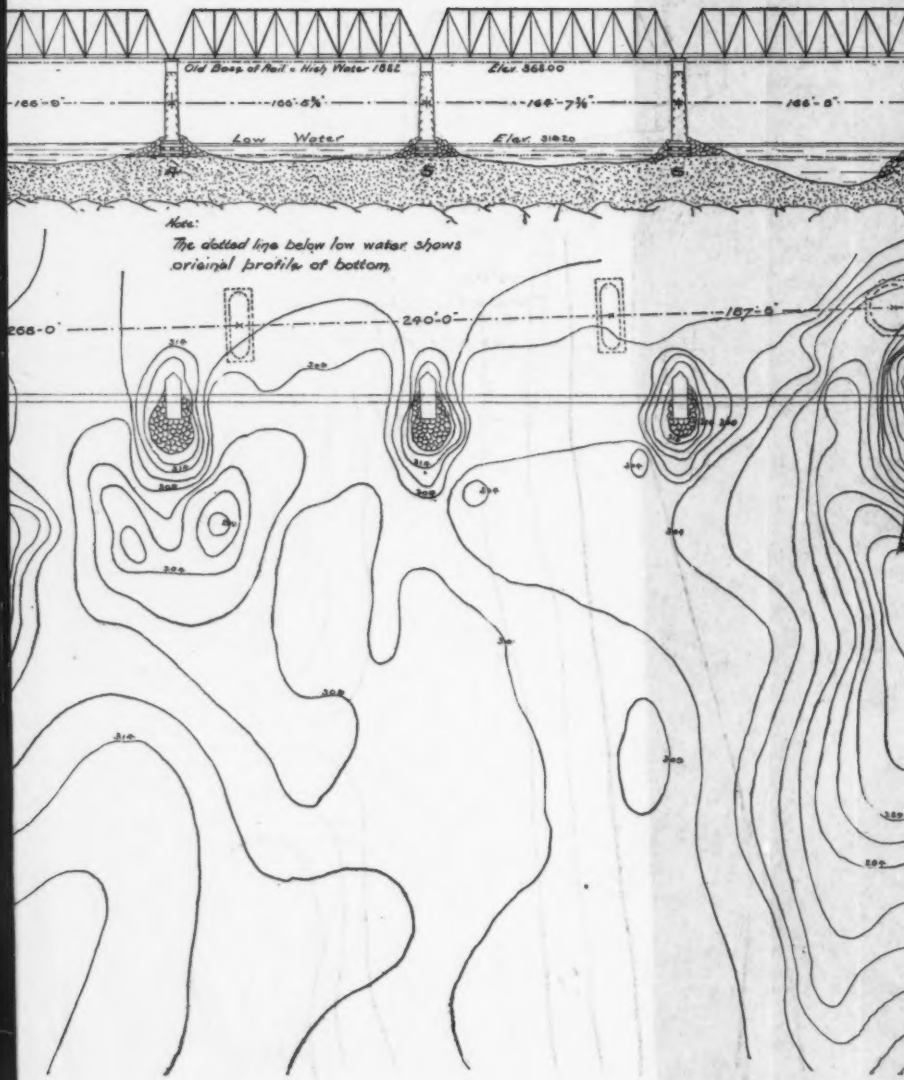
"The abutment on the east side is founded on rock. I was told by the Chief Engineer that the company was not willing to go to the expense of sinking coffer-dams down to solid rock."

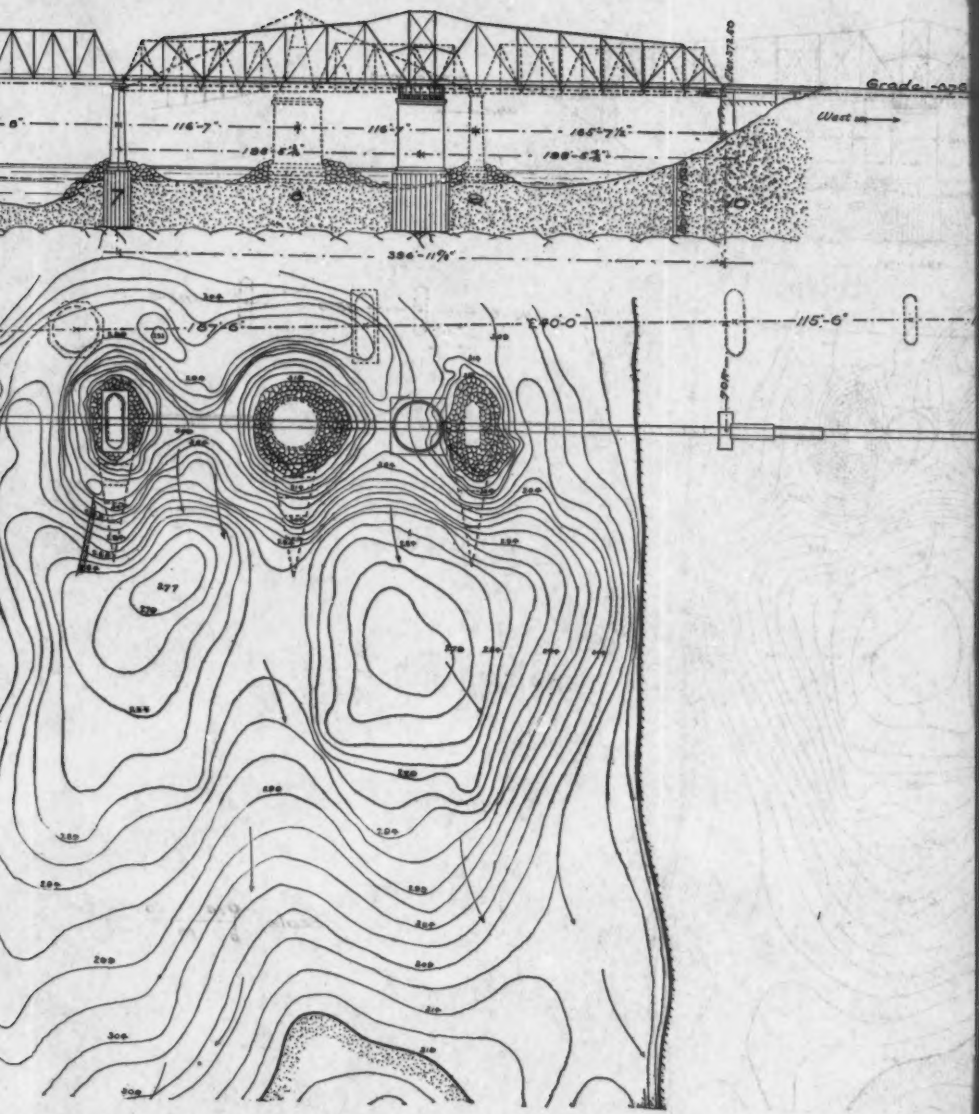
Mr. J. R. Bryan, Superintendent of Bridges on this division, who was employed as a carpenter in the preparation of the foundations of piers Nos. 5 and 6 (see Plate XXVI), in a report to the Chief Engineer dated March, 1890, describes the work done in about the same manner as Mr. Ball, except that a floor was laid on top of the crib to receive the masonry and that the cribs were not filled with stone.

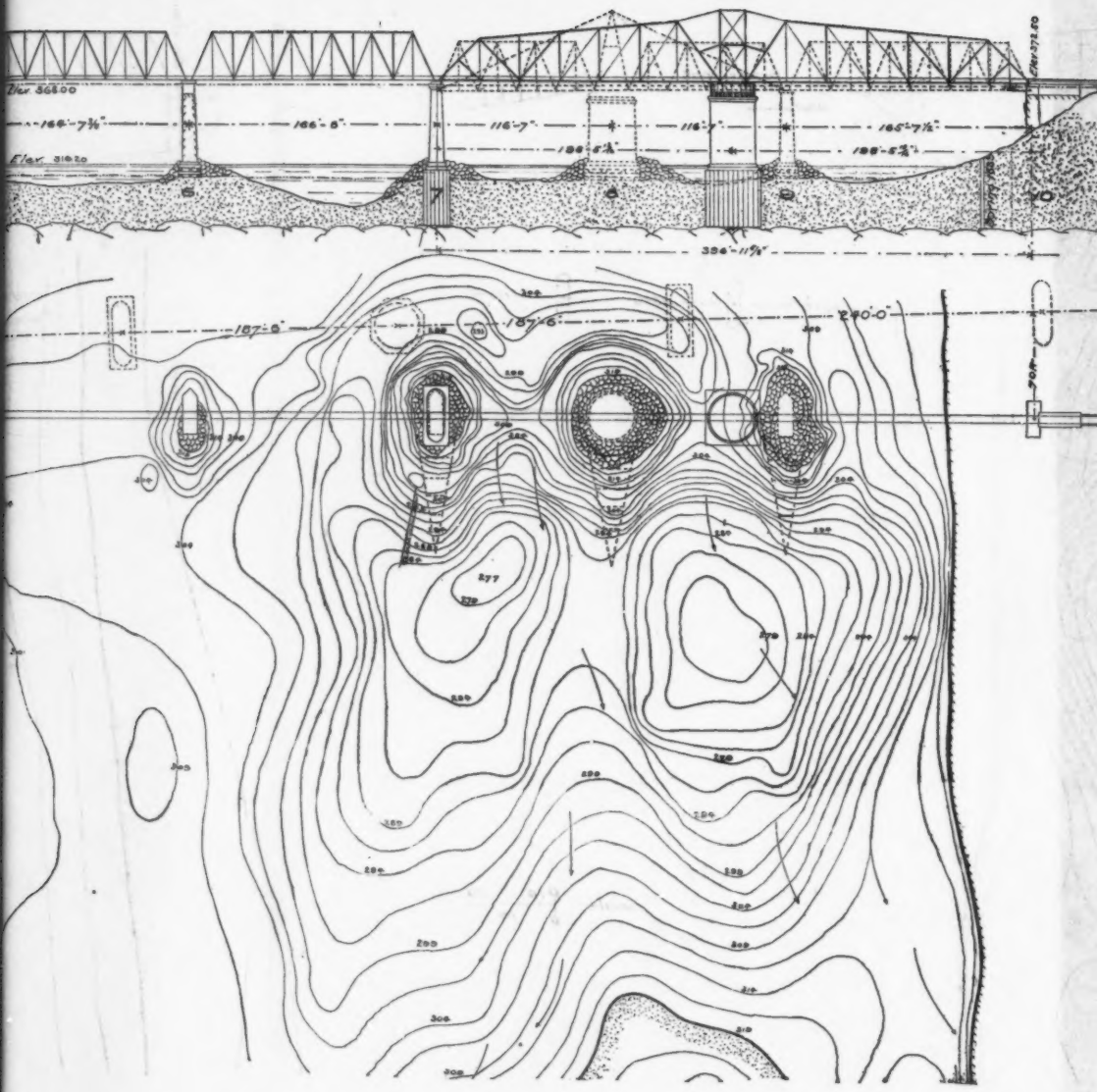
With regard to the condition of the bridge at the time of making the report he says:

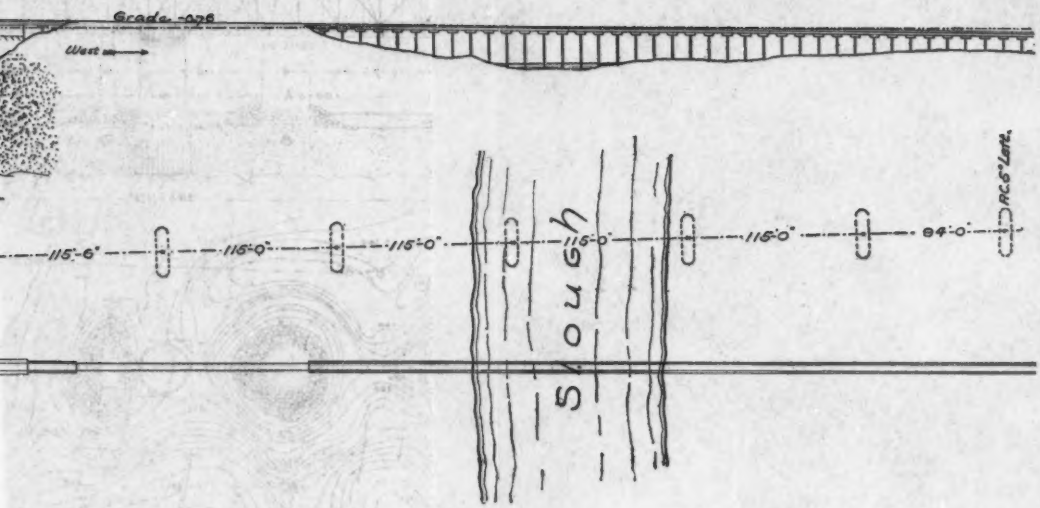
"Pier No. 7 has settled and the top has moved toward the round pier 1 ft. 7 ins., the pier being plumb on the west side. Two hundred car-loads of rock have been thrown around this pier. In the rise of 1882, a large log house lodged against pier No. 9, and it settled eastward about 4 ins., down stream 8 ins., and vertically about 3 ins. When

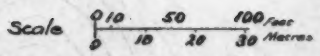

Grade +9.75
Elevated
East
182'-7½"
188'-2½"
166'-0"
Old U.S.
Transport
Hulk
268'-0"
60'-0"


Old Bear of Rail = High Water 1882
Elev. 568.00
Low Water
Elev. 510.20
Note:
The dotted line below low water shows original profile of bottom
266-0"
240'-0"
187-8"


116'-7"
116'-7"
165'-7½"
West
396'-11⅞"
187'-6"
290'-0"
115'-6"


Elev. 568.00
Elev. 318.20
166'-7½"
166'-8"
116'-7"
116'-7"
165'-7½"
198'-5½"
198'-5½"
396'-11⅝"
187'-6"
187'-6"
280'-0"
704'
Elev. 372.20
Elev. 318.20
5
7
9
10


Grade -0.28
West
115'-6"
115'-0"
115'-0"
115'-0"
115'-0"
84'-0"
Slough


Scale
0 10 50 100 Feet
0 10 20 30 Metres

the water fell it showed that five large stones at the north end of the pier at low water-mark had fallen out. These were replaced by the company's masons, and under the direction of the Chief Engineer cribs constructed according to sketch (Fig. 1) were sunk on the down-stream

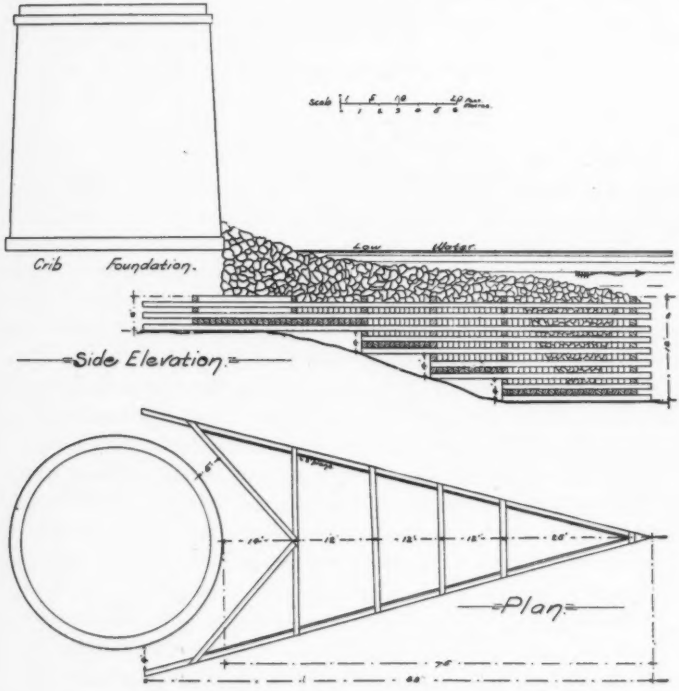


FIG. 1.

side. One hundred and fifty car-loads of rock and slag were thrown around it. Since then the pier has settled 1 in. eastward, and 2 ins. down stream. Similar cribs were sunk at piers Nos. 7 and 8. Pier No. 8, the round pier, has settled down stream about 4 ins. At the lowest water-mark a few inches of the timber in the cribs of all the piers is exposed to view. Up to date 843 car-loads of rock have been thrown around all the piers."

In 1871, the Nashville, Chattanooga and St. Louis Railway acquired control of the Nashville and Northwestern Railroad, and Col. R. C.

Morris, M. Am. Soc. C. E. (deceased), took charge of the bridge. In a report on its condition in 1890, he says:

"Unmistakable evidences of settlement were visible in 1871. A small island of gravel below the draw was then visible which has been increasing to the present time and now contains 5 acres above low water. A few years later another island was visible at low water near the east side and now contains ¼ acre. The round pier was pointed in 1871, 300 barrels of grout poured into it and bands put around it to prevent further spreading.

"On February 25th, 1890, spans Nos. 4–5 and 6–7 were blown away by a tornado. The spans were staged with piles at once, although the river was quite high (see Hydrograph, Fig. 4), and new spans put in, built on the same plan as the old ones, except that the floor-beam hangers and suspenders were strengthened and the lateral system rearranged."

Continuing, he describes the condition of the bridge and foundations, urges immediate action and presents plans and estimates for a new structure to be built just above the old one and estimated to cost $485 000. This bridge was to consist of two 60-ft. deck girders, two 268-ft. river spans, two 240-ft. river spans, one 375-ft. draw, twelve 115-ft. and one 94-ft. shore spans. The shore piers were to be made of iron shells filled with concrete and founded on piles. The river piers were to be of masonry, founded on pneumatic caissons. The proposed bridge is shown in dotted lines, just above the old bridge on Plate XXVI. This plan was approved by the management, and an agreement entered into with the Government to complete the new bridge and remove the old one by November 15th, 1894.

On account of continued decrease in earnings, the letting of the work was postponed from time to time, and at the time of the death of Colonel Morris, in November, 1892, nothing had been accomplished.

On November 8th, 1892, the writer was appointed Chief Engineer. The financial condition of the country and the physical condition of the bridge had both reached a serious point. The process of undermining the piers was going steadily on, and the islands below the piers were growing constantly larger by the addition of gravel from the scoured places.

Since 1879 the writer has been a close observer of the condition of this bridge, making frequent soundings to determine the depth and scour. The contours of the bottom, shown on Plate XXVI, are the result of soundings made in the fall of 1893, except those above con-

tours 311 and around piers Nos. 7, 8 and 9, which are from soundings made in 1892, before any dredging had been done around these piers.

The holes below the piers are deeper than they ever were before, but in previous soundings the depth varied, being sometimes 8 or 10 ft. shallower than the year before. The deepest scours appear after the heaviest floods, when the current attains a velocity of about 5 miles an hour, and during a season of light rises the holes appear to fill up. The islands, however, never get smaller. The gravel, which in light rises is washed from above and around the piers into the holes below, is again taken up and held in suspension by the heavier rises and stronger whirlpools formed below the piers until deposited on the islands below.

Steamboat men had for years been complaining of the narrowness of the channel and the danger to boats in passing through the draw. The Government, in order to compel early action in the matter, had ordered that no more riprap be placed in the river at that point and was prepared to enforce this order by indictment under recent acts of Congress, making such offences punishable.

The heavy expense of the proposed new structure and the time required for its completion put it beyond consideration, and a cheaper solution of the difficulty was necessary. It was apparent that the heavy scour around the draw piers was caused by the contracted channel, the scour around the other piers being comparatively light, except around pier No. 3. The channel here is contracted by a number of old United States transport and gunboat hulls, strewn along the east bank above the bridge, which were sunk by General Forrest with his artillery on the opposite shore during the late war.

Careful observation has failed to discover any settlement in any of the piers except Nos. 7, 8 and 9. Believing that if a good channel were provided for boats on the west side, no objection would be raised to riprapping or sinking mattresses around the other piers, and that the widening of the channel on the west side would decrease the scour around the other piers, it was decided to hold all piers east of No. 7, to strengthen, as far as possible, the old superstructure, and to put in a new draw at the west end.

It was considered inadvisable to found the new piers on gravel so near the large holes that had scoured out nearly to the rock, and therefore it was decided to found them on bed-rock. The question of

how close to the old piers the new ones could be placed was carefully considered by Col. Morris in his investigations. Opinions were sought by him from other engineers with the following results. One of these said:

"Material is sometimes disturbed 100 ft. from caisson during sinking. I would locate piers at least this far, if possible."

Another said, "Under favorable circumstances, I will be willing to go as close as 15 ft. to the old pier, provided that I can guard against danger by driving piles to support the ends of the spans resting on the pier. I would prefer, however, to keep 25 ft. or more away, as I could secure the safety of the old pier at that distance with less trouble and expense."

Another said, "I would advise, in sinking your piers, to get them at least 25 or 30 ft. from the foundation of the old piers. In the first place, sinking through rip-rap will be troublesome, especially as you will encounter more on one side of the caisson than on the other, and, if possible, you should be far enough away to avoid striking the rip-rap at all. Another reason is, that owing to the weight of your present pier, if you were to sink very close to it the pressure would be unequal on the caisson, and it would have a tendency to move away from the pier."

Realizing that to disturb the round pier meant an early stopping of navigation, and that, even if pier No. 9 was disturbed, the span could be carried either on trusses up and down the stream, or on piles, the arrangement of spans shown on Plate XXVI was recommended, and on February 23d, 1893, was ordered carried out.

The approval of the Government was secured, the agreement for a new bridge annulled and a new one entered into, providing for the completion of the new draw on January 1st, 1894.

Before asking for proposals, borings were made near the sites of the new piers in the following manner: A pile-driver with short leads was run on the bridge and old 6-in. Phœnix columns were driven as piles, being coupled in 30-ft. sections by means of eight iron straps, bolted through the holes left by the rivets, which were cut out. As this iron pile was driven down, the inside was pumped out with an iron-valve bucket. Previous borings at the shore indicated that rock would be found at elevation 272. Before the lower end of the pile had reached this elevation it became impossible to lower the gravel and sand inside the column on account of the upward flow caused by the pressure from without. It was then decided to drive the column

down without pumping out. This was done, and the lower end appeared to stop at a point 6 ft. below the elevation at which rock was expected. Being suspicious of this, a 5-in. wrought-iron pipe was pumped down inside the column, and rock was found at elevation 275. The cause of the difference was revealed when, on landing No. 9 caisson, one of the flanges of the column, which had been driven quite close to the edge of the proposed caisson, was found turned up under the cutting edge. The other column was driven to rock without trouble.

The plan contemplated the sinking of two foundations to bed-rock for the new pivot pier and pier No. 7, the tearing out of the masonry of Nos. 7, 8 and 9; the dredging of the channel to a depth of 8 ft. below low water, the carrying of the old spans while work was in progress, the erection of the new draw, the removal of the old draw and one fixed span, the renewal of all wooden truss members in the old combination spans, except spans Nos. 4–5, 6–7; the placing of equalizers under all cross-floor beams on the old spans, putting in heavier pier boxes, putting in new floor-beam hangers and suspenders on all the old combination spans, except 4–5 and 6–7, the rearrangement of the lateral bracing and the raising of the entire bridge 4½ ft. to get above the high water of 1882.

Proposals were invited for the work as a whole and in four parts, the foundations, the masonry, the superstructure of the new draw and repairs to the old bridge.

A plan for carrying the spans resting on pier No. 7, by means of lateral trusses 60 ft. long, resting on temporary pile piers, was suggested in the invitation.

Contractors were required to furnish plans for the foundations proposed and the manner of carrying the old spans while sinking was in progress. The pneumatic process of sinking was proposed, but others, such as dredging through open wells, would be considered, if accompanied by sufficient evidence of proving successful.

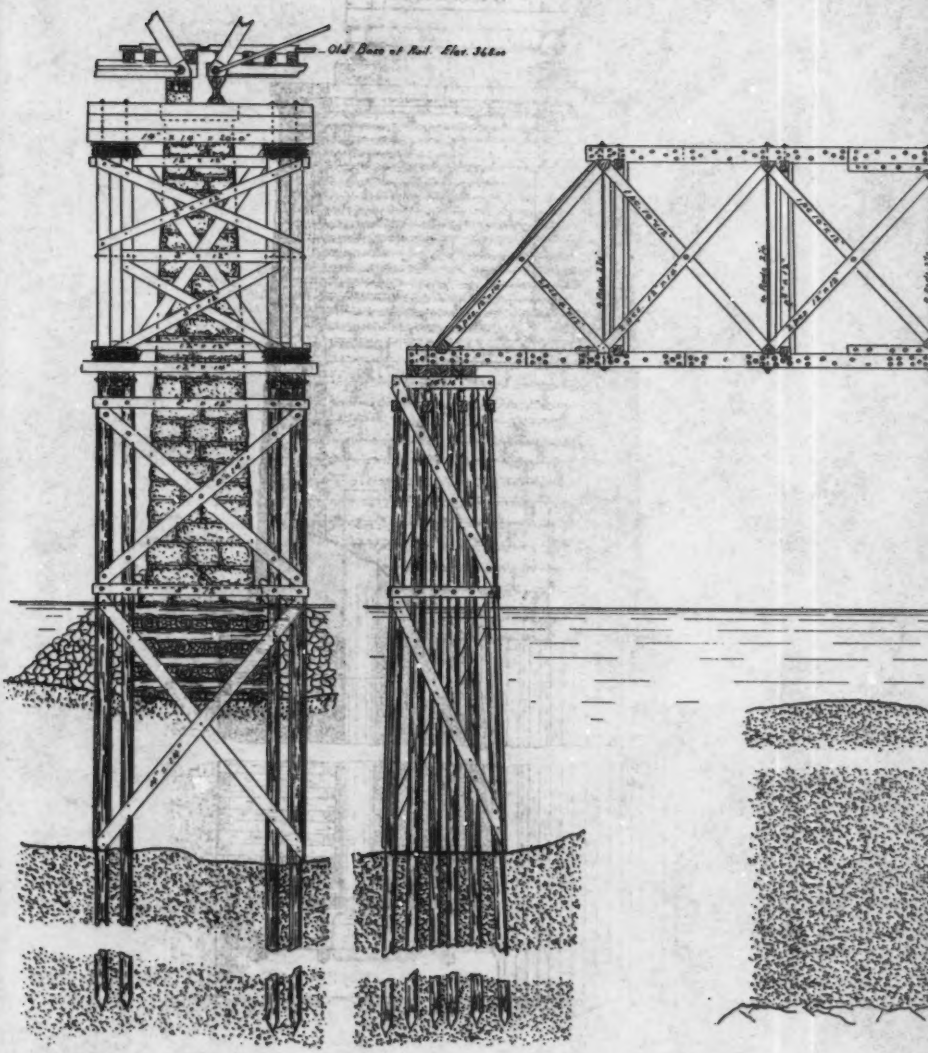
On April 28th, 1893, the work of substructure and masonry was awarded to the H. S. Hopkins Bridge Company, and the entire superstructure to the Louisville Bridge and Iron Company, both the lowest bidders.

The plan for carrying the spans and the caissons and cribs were substantially as shown on Plate XXVII.

The pivot pier caisson was 36 ft. square and 16 ft. high, and the one for pier No. 7 was 40 ft. 6 ins. by 16 ft. 9 ins. and 12 ft. high. Cribs filled with concrete rested on the caissons and extended to within 2 ft. of low water. All were designed by Mr. Frank D. Moore, Vice-President and Engineer of the H. S. Hopkins Bridge Company. The details of the caissons and cribs were practically the same as those designed by Mr. Moore for the Quindaro Bridge, described by him in his paper published in *Engineering News*, April 4th, 1891, the principal points of difference being that the sides of caissons were plumb, the abutting timbers lapped alternately, instead of being halved and lapped, and a steel shoe was used on the cutting edge. Mr. Moore gave the work his personal supervision from its beginning until the masonry was nearly completed.

In June Mr. J. H. Pope was placed in charge of the work as Resident Engineer, and remained until the completion of the contracts. In July W. H. Gahagan, Assoc. M. Am. Soc. C. E., took charge of the work as superintendent for the contractors. J. M. Johnson, M. Am. Soc. C. E., President and Engineer of the Louisville Bridge and Iron Company, designed the new draw-bridge, and Mr. John E. Gould had charge of the erection.

In order to secure the safety of the draw against floods, it was decided to sink the pivot caisson first, the contractors believing it more economical and safer to sink only one at a time. Believing it impossible to sink the pivot caisson so close to old No. 9 pier without endangering it, it was the intention to erect the carrying spans at No. 9 first, and take it down before sinking began. The plan was objected to by some of the members of the contracting firm as too expensive and unnecessary. Accordingly it was agreed that if piles could be driven close to No. 9, on each side to bed-rock, that this should be done and the spans supported on them. The experiment of driving one pile was to be made before starting others, but contrary to this agreement, eight piles were started down on the west side and each driven about 6 ft. That night, about 12 o'clock, the pier careened, the top moving westward about 4 ins. and down stream about 1 in., buckling the bottom chord of span 9–10, and displacing the lifting gear of the draw. The pier was now plumb in the axis of the bridge. It was deemed unsafe to experiment further in this direction, and accordingly steps were taken at once to erect the carrying spans as first proposed.


Old Base of Rail Elev. 342.00

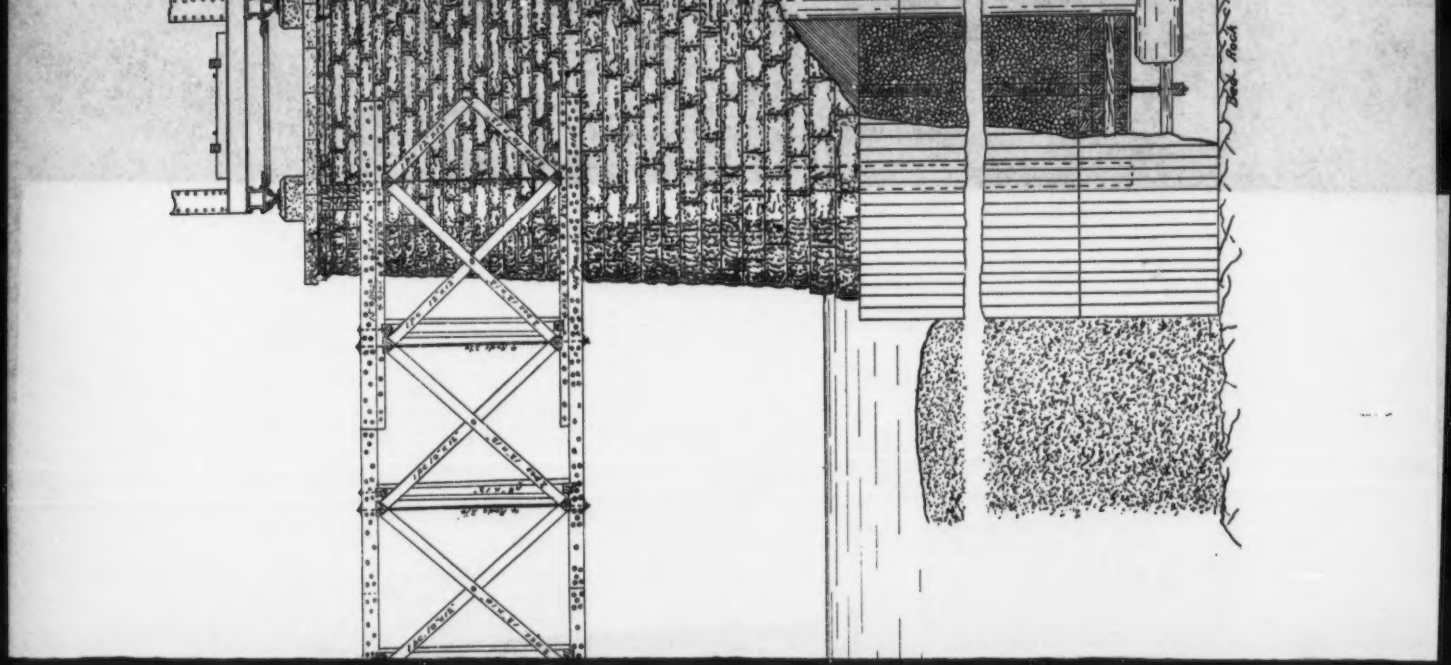
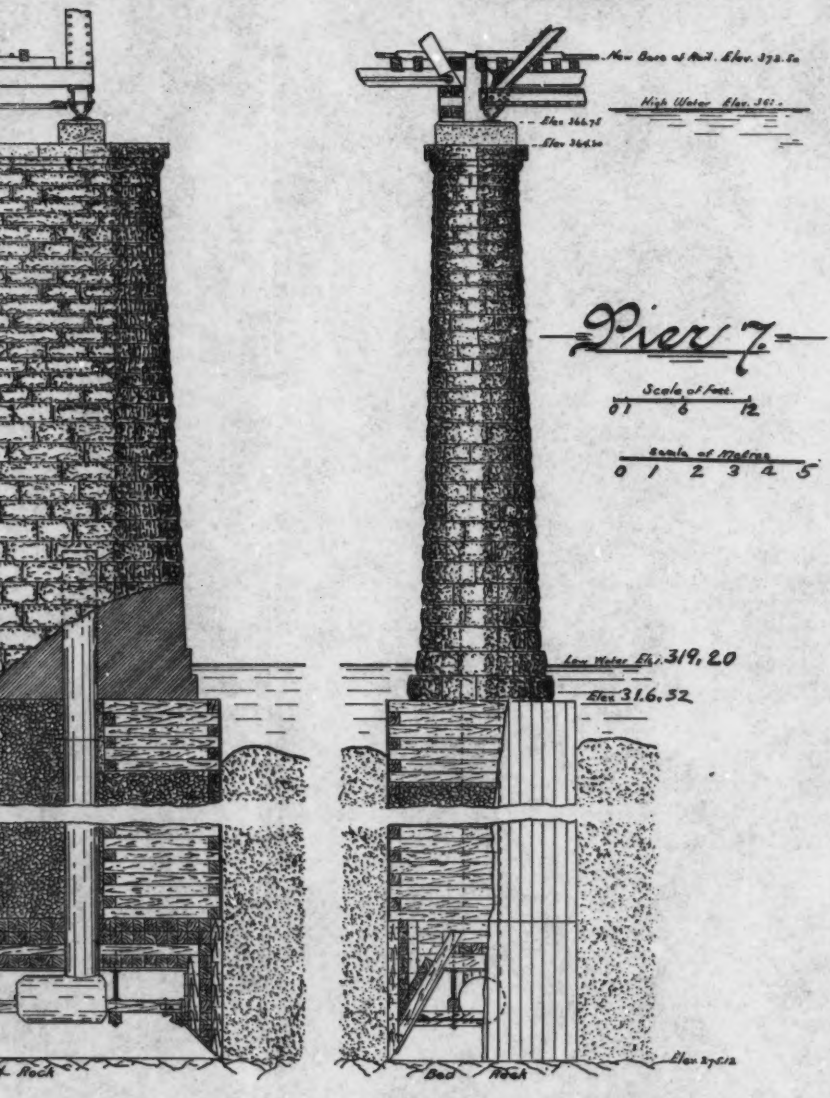


PLATE XXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXIII, No. 744.
McDONALD ON JOHNSONVILLE BRIDGE.



This was done, and the old pier taken down nearly to low water before sinking was begun, on August 15th.

Fearing that the weight of the old pier and the sloping rip-rap would tend to displace the caisson eastward, it was started 1 ft. too far westward. Notwithstanding the fact that the rip-rap had previously been partially leveled by means of a weighted toothed drag scraper, the caisson had moved to a point 1 ft. too far east before its east edge had begun to penetrate. Shores were placed between the caisson and the round pier, and a heavy bank carried under the west cutting edge. In spite of all efforts, when the caisson was righted up on the rock, the top was found to be 1½ ft. too far east.

When the cutting edge reached elevation 287, the masonry was begun, the weight of the finished crib being insufficient to sink it further. The first course of masonry was set within 6 ins. of the west edge of the crib, the original plan being to place it 2 ft. from the edge all around. When the cutting edge was landed and the masonry leveled, the pier was found to be exactly in position east and west, but 2 ins. too far up stream. This was corrected as the masonry was built up, and the top was finished in exact position.

After penetrating all the rip-rap at elevation 299, nothing but gravel and sand was found until elevation 291 was reached. Here a 4-in. stratum of river gravel was reached, closely cemented with sand and iron. Fragments of this stratum are to be found in many places on the island below. Nothing else but gravel was found save a 4-in. stratum of blue clay directly on the rock. Measurements taken from the water at the four corners showed the rock to be level.

The sinking of the caisson had disturbed the gravel around it very little. Nearly all the false-work piles used in the erection of the carrying span were left erect, only the four piles between the caisson and the old pier showing any signs of disturbance, these leaning slightly towards the caisson. None of these piles had penetrated more than 15 ft.

As soon as the caisson was landed piles were driven between the carrying trusses, forming a pier to carry the old spans. The supporting trusses were then removed to the site of pier No. 7 and re-erected. It being apparent that the trusses were longer than necessary, and they having sagged under their former load on account of the rough framing and shrinking of the green gum timber in the chords and angle

blocks, and the piles previously driven for the piers having found so little footing on account of the deep scour, it was decided to shorten the trusses before erection by taking off the end panels. New piers were then driven and the shortened trusses erected thereon.

As soon as the weight of the old structure was transferred to the trusses, the old pier was taken down. A steam dipper dredge was put to work clearing away the rip-rap and old cribs, which was done with little difficulty except that the space for handling the dipper was limited by the trusses.

The cypress and poplar logs taken out of the crib floated off as they were released. All the timber was sound. There was no flooring found on top of the crib, and the logs were, with few exceptions, round, being notched and drift-bolted at points where they crossed. The sinking of this caisson and completion of the masonry was quickly accomplished. Little rip-rap was encountered, the dredge having removed nearly all within the limit of the caisson. The stone was set between the trusses by means of a traveler at the level of the top chord of the old bridge, which took the stone directly from the cars standing on the bridge. This occupation of the track, interfering with the erection of the superstructure, necessitated a great deal of setting at night. Some heavy freezes occurred before this masonry was completed, and some of the stones which had been lately quarried cracked before being set. Others were substituted for them. No stone ever cracked after being set, and all are perfect at the present writing. This stone has since been adopted by the Government engineers for locks on the Cumberland River improvement. It is Niagara limestone and was taken from the quarry at Newsom on this division of the Nashville, Chattanooga and St. Louis Railway.

The concrete, composed of crushed limestone, 5 parts, screened Hollow Rock sand, 2 parts, and Louisville cement tested by Mead & Shaw, 1 part, was mixed on the shore, hauled out on hand cars, and dumped through chutes hung to the bridge by ropes. The fall of concrete was directed to any point desired by means of hand lines attached to the chutes. Little ramming was required.

The pressure-house was placed on the shore at the end of the bridge and connected by pipes with the caissons. Turning the draw did not interfere with the pipes to the pivot pier, and as pressure was not put on No. 7 caisson until after navigation was closed, there was no trouble here either.

Up to the time of the completion of the pivot pier, navigation through the draw had not been interfered with, but on October 20th the removal of the old draw began and the traffic of the river was transferred from boat to boat by locking them together under the bridge.

Span 9–10 had already been staged and removed. The west end of the draw was raised first, by a traveler working from the center, the staging and removal of the old draw going forward also. The east arm was then raised and the draw turned for boats on December 4th. It will be remembered that the base of rail of the new draw was to be $4\frac{1}{2}$ ft. above that of the old one. The trusses of the draw were erected in their true positions on the new drum, the cross-floor beams and bottom lateral bracing being left out entirely. The track on the bridge was then raised on the false-work to the new grade and the floor system put in, the approaches, including the two nearest spans of the old bridge, being raised at the same time. The traffic of the railroad was kept up continuously, no delays of any note occurring.

It being important that the removal of the old pivot pier should be completed before the river rose, as soon as the east arm of the draw was connected and before the track was raised and the floor system put in, temporary floor beams of wood were swung from the bottom chord by means of U-shaped hangers made of 3-in. round iron passing through 12 × 12-in. sticks laid along the bottom chord and supported on blocks near the panel points, and the track, which had previously been blocked up on the old round pier, was carried across it clear.

The removal of this pier was quite difficult at first, the grouting having proved very effectual for a distance of about 20 ft. from the top. Below this point the silt deposited in the piers by the frequent high waters had almost filled the voids, and, when not entirely filled, the coating of mud on the rocks prevented the cement from taking hold. The foreman who set the stone on this pier has recently stated that no mortar was used in it at all. The timber crib under it was found to be as above described. The dredging of this crib and the mound of rip-rap around it to a depth of 8 ft. below low water was finished and the whole substructure plant removed from the work by December 20th.

The strain diagram (Fig. 1, Plate XXVIII) shows the principal features of the superstructure. The total weight of the draw, exclusive of the turning engine and gearing, is 734 809 lbs.

The turning arrangement consists of two 8 × 10 reversible Atlas engines, with provision for turning by hand. The ends are raised and lowered by screw jacks, operated by a single wrench at each end.

The strain sheet (Fig. 2, Plate XXVIII) is of the old combination spans which still remain, the left half showing stresses produced by the loads for which it was designed, and the right half those produced by the loads now running over it, which from assurances received are the heaviest which will be placed upon it by engines for some years.

The timber in all the trusses, except those blown away in the tornado, had been in place since 1871, and it was feared that in spite of the careful housing some of it was rotten. This was all renewed while the masonry was being built. There was not a dangerous stick found under the housing, and all were in a remarkable state of preservation. The floor beams in these old spans were originally supported by castings carried on pins on the ends of the hangers and suspenders. It was impossible to adjust them, and the deflection in the floor beam caused the load to come almost entirely on the inside member. This was remedied by placing equalizers built of channels (see Fig. 3, Plate XXVIII) at the bottoms of the suspenders and hangers. Fearing that the continual bearing of the loads by the inside hangers and suspenders had weakened these members, they were replaced by new ones. The lateral system, which was very poor, was also rearranged, as shown on Plate XXVIII. The old spans were all raised to the new grade on timber blocking.

The railway completed the work of topping out the old piers. Bents were set up on each side of each pier, founded on rip-rap, and both spans carried on stringers thrown across the piers. The old coping was first removed, the walls built up and the coping put back. These spans are carried now as formerly on 2 ft. of timber on the tops of the piers, it not having been decided yet whether, when these spans require to be made heavier, new piers will be built or new metal spans be built on the old piers. All the old piers were carefully pointed in order that any settlement might be quickly detected. No cracks have so far appeared in any of the pointing.

All the iron work received one coat of red lead and oil before shipment, and after erection another coat of the same, with the addition of 1 oz. of lampblack to each pound of red lead.

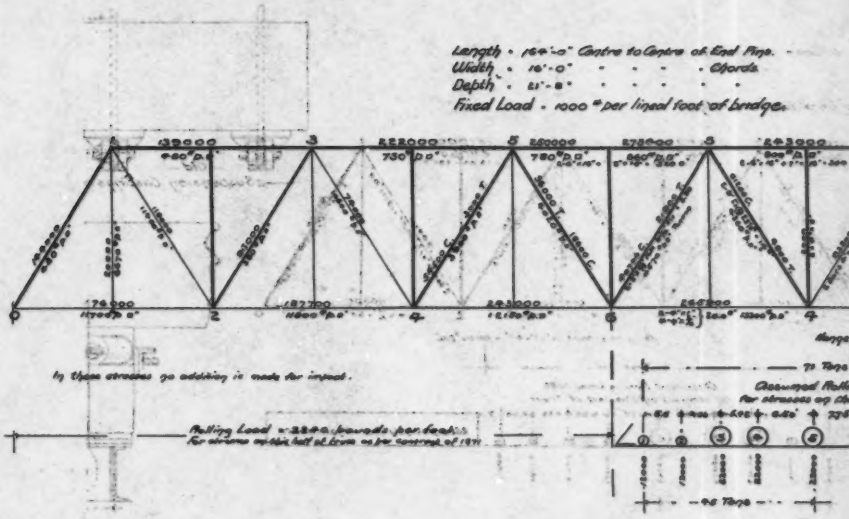

Length - 160'-0" Centre to Centre of End Pins.
Width - 18'-0" - - - - Efforts
Depth - 21'-8" - - - - -
Fixed Load - 1000 ℔ per lineal foot of Bridge.
In these stresses no addition is made for impact.
Rolling Load = 2240 pounds per foot
for stresses on this half of Truss on the covering of 1911

FIG. 2.

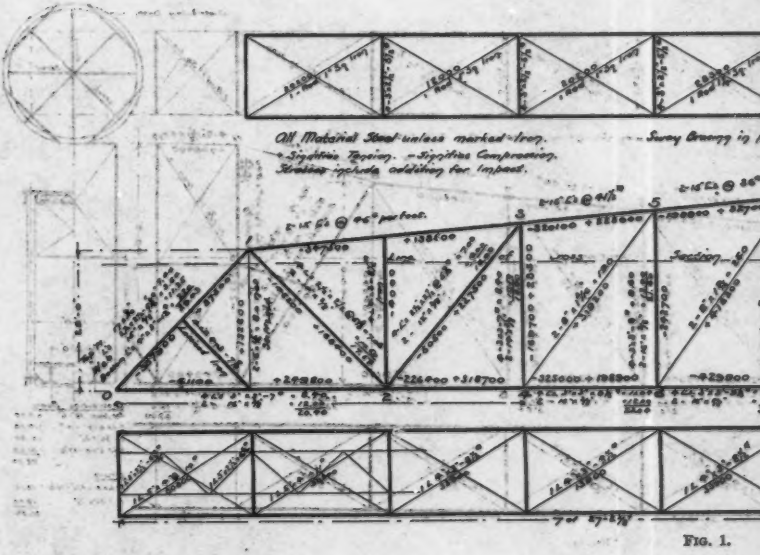

All Material Steel unless marked Iron.
Stresses include additions for Impact.
Junction

FIG. 1.

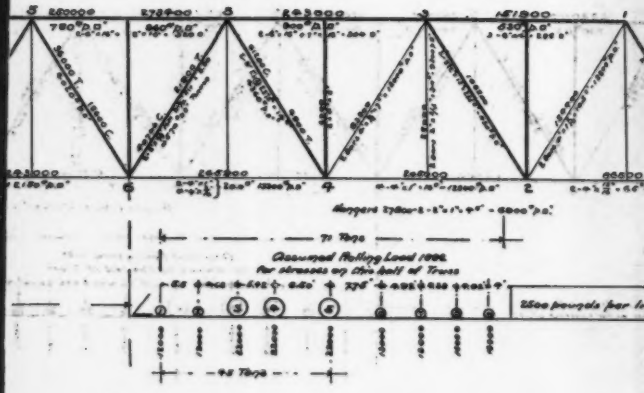

Centre to Centre of End Pins.
Chords.
Load = 1000 ᵗᵒⁿˢ per lineal foot of bridge.
Assumed Rolling Load 1000
for stresses on One half of Truss
2500 pounds per

FIG. 2.

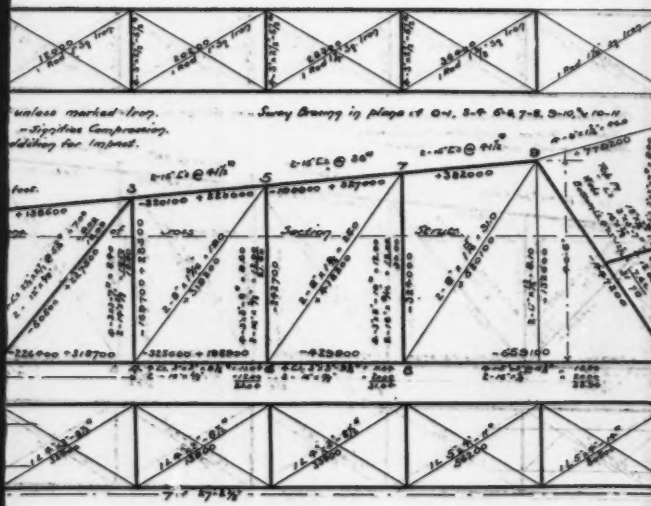

unless marked Iron.
Signifies Compression.
Section
Struts.

FIG. 1.

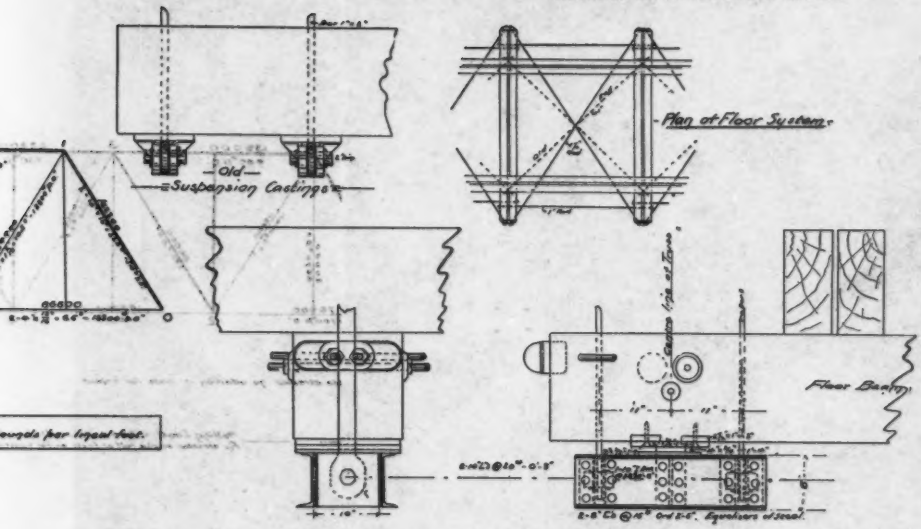
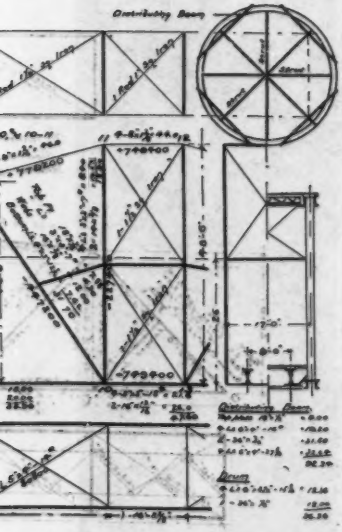


FIG. 3.



Length 386'-11⅝" Centre to Centre of End Pins.
Width 17'-0" " " " Chords.
Depth 28'-0" at ends, 4-0"-6" at 9, 4'-6" at centre.
Fixed Load 2100 pounds per lineal foot of bridge.
Rolling " 2-120 tons Consols + 3500 # per lin. foot

Stringer:-

Top Plate	12"×⅞"	= 0.60
" Angles	6"×6"×⅞" @ 19#	= 0.96
Bot. Plate	12"×⅞"	= 0.60
" Angles	6"×6"×⅞" @ 19#	= 0.96
Web	30"×⅞"	= 11.26
		27.05

Floor Beams:-

Top Flange	2-3½"×3½"×½" @ 9⅝#	= 4.10
Bot "	2-3½"×3½"×½" @ 9⅝#	= 4.10
Web	96"×⅞"	= 8.00
		39.20
Cross Ties	8"×11½"×13'-0"	

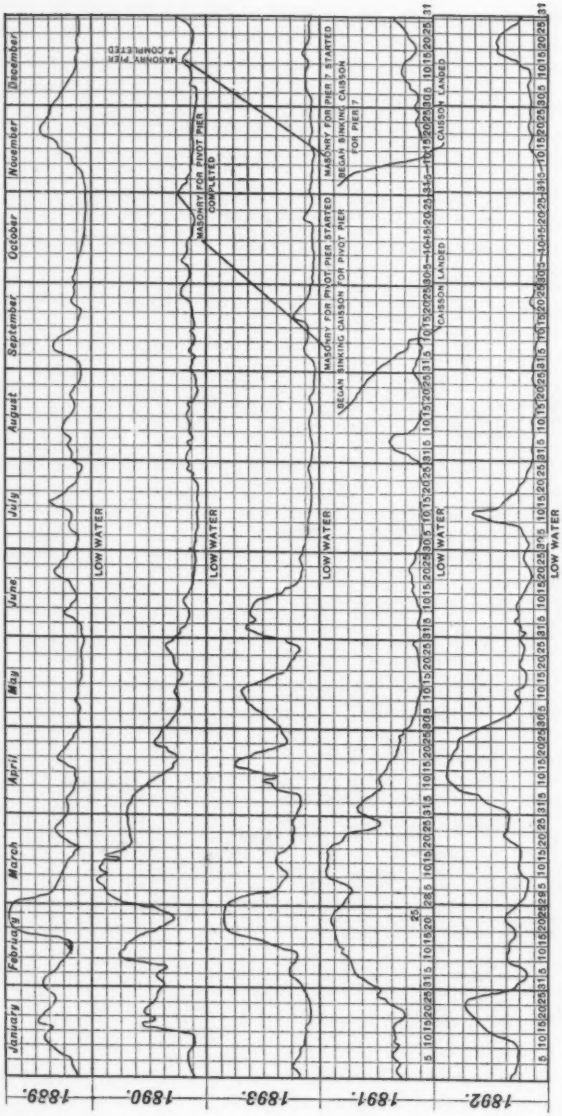
Dead Load = Extra weight of Truss & Floor. Timber 54# per Cu. foot. Rails & fastenings 60# per lineal foot of track.

Wind Load = 30# per sq. foot on exposed surfaces of both trusses and on moving train surface of 18 sq. feet per foot of track applied 8 feet above the rail.

Effect of Impact to be added to Maximum Stresses resulting from engine and train load by the formula—

$$I = S\left(0.7 + \frac{4}{L}\right)$$

where S = Calculated maximum live load stress.
I = Effect of impact.
L = Length of loaded distance in feet.



HYDROGRAPH.

NOTE.—Distance between horizontal lines = 5 ft.

FIG. 2.

The east approach was an embankment and was raised by filling. The west approach consisted of a short bank at the end of the bridge and a cedar pile trestle with five piles to the bent. Wherever the raise was less than 12 ins., it was accomplished by doubling the caps, but above 12 ins. additional brace piles were driven on each side at each bent, and the old piles spliced by setting blocks of cedar on them and cover jointing.

On Fig. 2, page 183, is shown the progress of sinking the caissons and building the masonry, also the stages of the water at this point from 1888 to the close of the work.

The cost of the entire work is shown in the following statement:

SUBSTRUCTURE.—COST OF PIVOT PIER.			
Caisson.			
119 792.40 ft. B. M. timber in caisson, at $38		$4 552 11	
95 727.50 ft. B. M. timber in crib, at $28		2 680 37	
54 975.00 lbs. iron in caisson in crib, at 4 cts.		2 199 00	
96.00 lineal feet shafting left in place, at $7		672 00	
44.00 lineal feet sinking below low water, at $344 81		15 172 42	
313.40 cu. yds. material removed through lock, at $35		10 969 00	
1 085.90 cu. yds. concrete in crib and pockets, at $6		6 515 40	
233.50 cu. yds. concrete in air chamber, at $12		2 802 00	
Cost of caisson		$45 562 30	
2 112 cu. yds. displacement, average cost per cubic yard	$21 57		
Cost per vertical foot (44 ft.)	1 035 50		
MASONRY.			
415.62 cu. yds. face stone, at $12		$4 987 44	
725.19 cu. yds. backing, at $7		5 076 33	
24.52 cu. yds. coping, at $16		392 32	
Cost of masonry		$10 456 09	
Less credit old stone taken out		1 065 00	
Net cost of masonry		$9 391 09	
Total cost of pivot pier			$54 953 39
COST OF REST PIER.			
Caisson.			
54 950.00 ft. B. M. timber in caisson, at $38		$2 088 10	
60 097.00 ft. B. M. timber in crib, at $28		1 682 72	
40 964.00 lbs. iron in caisson and crib, at 4 cts.		1 638 56	
108.00 lineal feet shafting left in place, at $7		756 00	
44.08 lineal feet sinking below low water, at $217 01		9,565 87	
17.00 cu. yds. material removed through lock, at $35 00		595 00	
564.20 cu. yds. concrete in crib and pockets, at $6		3 385 20	
108.00 cu. yds. concrete in air chamber, at $12		1,296 00	
Cost of caisson		$21 007 45	
1 107.51 cu. yds. of displacement, average cost per cubic yard	$18 96		
Cost per vertical foot (44.08 ft.)	476 35		

MASONRY.			
351.39 cu. yds. face stone, at $12	$4 216 68		
124.72 cu. yds. backing, at $7	873 04		
15.78 cu. yds. coping and pedestals, at $16	252 48		
Cost of masonry	$5 342 20		
Less credit, old stone taken out	535 00		
Net cost of masonry	$4 807 20		
Total cost of rest pier		$25 814 65	
REMOVING OLD PIERS.			
1 892.90 cu yds. old masonry removed, at $1 75.	$3 312 57		
3 393.30 cu. yds. removing old cribs and dredging, at $5 32	18 068 90		
Total cost		21 381 47	
Supporting old spans	$4 900 00	4 900 00	
Freight and passage (Company service)	$4 966 13	4 966 13	
REPAIR OF OLD MASONRY.			
Pointing, grouting and stone work	$4 630 83		
Supporting spans for topping out piers	826 38		
Total cost		5 457 21	
Total cost of substructure			$117 472 85
SUPERSTRUCTURE.—RECONSTRUCTION OF OLD SPANS.			
Raising and repairing old spans	$10 641 00		
Freight and passage (Company service)	117 95		
Total cost		$10 758 95	
NEW DRAW.			
New draw, erected complete	$36 948 00		
Freight and passage (Company service)	117 02		
Total cost	$37 065 02		
Credit, by old spans taken out	4 755 00		
Net cost of new draw		32 310 02	
Total cost of superstructure			43 068 97
Raising approaches—			
Trestle-work		$3 141 27	
Grading		808 61	
Total cost			3 949 88
Transferring traffic while navigation was closed:			
Cost of transfer track and moorings		$886 82	
Damage to steamboat companies		3 300 00	
Total cost			4 186 82
Engineering—			
Preliminary boring		$998 88	
Superintendence		1 177 75	
Inspection		558 10	
Testing		542 67	
Surveys and drawings		182 00	
Total cost			3 459
Switching		$1 522 27	
Total cost			1 522 27
Grand total			$173 660 19

Recapitulation		
Cost of substructure	$117 472 85	
Cost of superstructure	43 068 97	
Raising approaches	3 949 88	
Transferring traffic	4 186 82	
Engineering	3 459 40	
Switching	1 522 27	
Grand total		$173 660 19

DISCUSSION.

Mr. Macdonald

CHARLES MACDONALD, M. Am. Soc. C. E., remarked that the subject treated in the paper is one which recalls many similar experiences where much is required to be accomplished with very little money. In this case the results seem to have been very satisfactory and reflect great credit upon the engineer in charge. It is an unpleasant thing to note that the allowance for engineering should be put down in the detailed statement of cost as only 2% of the whole estimated expenditure. It is to be hoped for the sake of the profession that the time will soon arrive when the remuneration of engineers will at least equal that of architects for the work which they have to do.

There is one question which he would like to have more fully discussed by those who have had experience in the sinking of caissons, viz., the lateral movement of the caisson in sinking.

He understood from the paper that the caisson was set 1 ft. out of center to the westward, in order to provide for a possible movement to the eastward in sinking, due to an excess of rip-rap on the high side, and that, as a matter of fact, the caisson did move 2½ ft. to the east before reaching bottom.

In sinking Pier No. 6, Hawkesbury Bridge, by open dredging, a similar movement took place. At the site of the pier the bottom of the river sloped diagonally across the long axis, and the pier moved bodily in the direction of this slope.

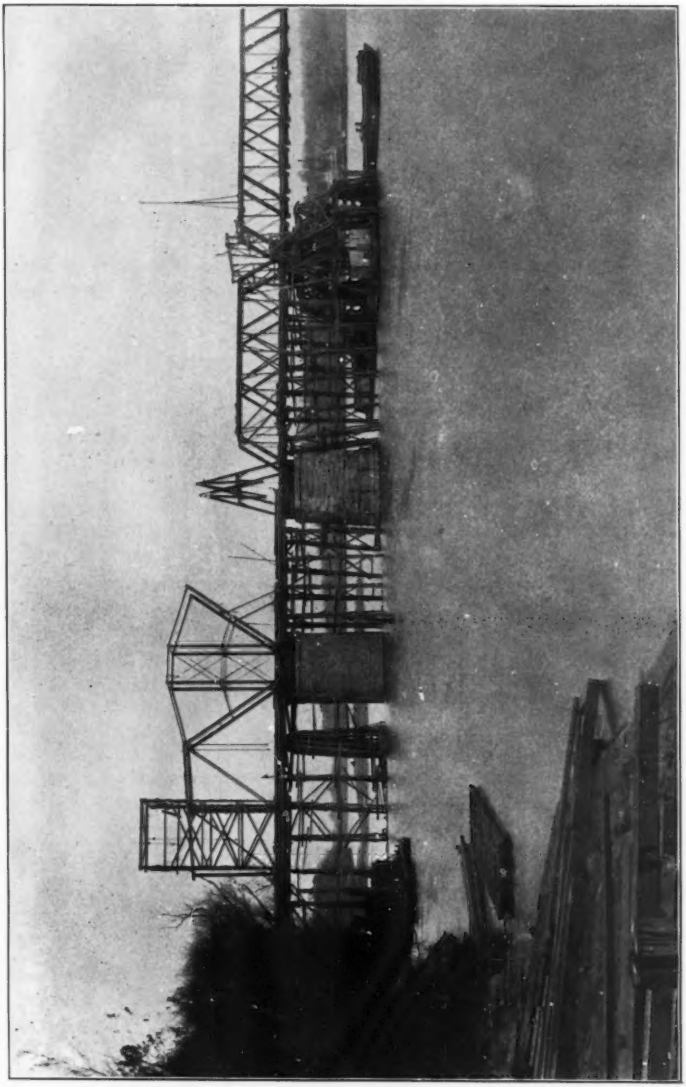
The movement was finally counteracted by depositing 2 000 or 3 000 yds. of rock close to the pier on the low side.

Mr. O'Rourke.

J. F. O'ROURKE, M. Am. Soc. C. E., said that the method employed to right the caisson at Hawkesbury, described by Mr. Macdonald, was precisely the reverse of that used to level one of the caissons of the Poughkeepsie Bridge, which were very large, measuring 60 x 100 ft. and 100 ft. high, and had two lines of dredging pockets.

These caissons, even after they were well down in the river-bed, had a tendency to rock, settling, perhaps, 3 or 4 ft. out of level, and, after they had been again dredged out and dropped, would tip the other way. It was then supposed that the high side penetrated deeper into the bed by virtue of the increased momentum gained in falling through a greater space. Accordingly the plan of removing less material from

PLATE XXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXIII, No. 744.
McDONALD ON JOHNSONVILLE BRIDGE.



under the high side was tried with caisson No. 5, which was 6 ft. out of level longitudinally and 3 ft. transversely, and when it was undermined sufficiently to move, it dropped 8 or 9 ft. In its new position it was only 0.4 ft. out of level longitudinally, and 0.1 ft. transversely. Mr. O'Rourke.

He thought the cause of the trouble with the pivot-pier caisson, mentioned by the author, must have been due to the fact that there was more rip-rap on one side of the pier than on the other. This would account for the movement, since in the case of rip-rap it takes place before the caisson is sufficiently embedded to hold it laterally.

The plan of simply lapping the timbers of caissons and depending upon drift bolts to hold them in place, was to be commended. The framing together of 12 x 12-in. timbers he thought a common mistake made by architects and engineers, and no more necessary than splitting bricks, to have them bond better at the corners of buildings.

In connection with the Poughkeepsie work, experiments were made to determine the adhesion of drift bolts in 12 x 12-in. timbers; 422 lbs. per lineal inch for 1-in. round bolts in $\frac{15}{16}$-in. holes was the minimum found, about 1 200 lbs. per inch being the average. It is therefore plain that an enormous force would be required to lift timbers fastened in this way, and still more would be necessary to slide them on each other.

T. C. Clarke, M. Am. Soc. C. E., in speaking of drift bolts, said, that in the original plans for the Poughkeepsie Bridge the caissons were designed before the Union Bridge Company took hold of the bridge to build it. They were then altered considerably. In the old plans all vertical timbers (some 25 or 30 sets) were to be drifted laterally in each course, in order to prevent the courses pulling apart. But as these vertical timbers were very awkward to connect in this way, and would take a great deal more time to build into the caisson, which was of wood, they were omitted; every course was put down horizontally in the water and without any cutting. Mr. Clarke.

It was necessary to ascertain what number and size of drift bolts would be necessary to prevent the buoyancy of the timber from floating the caissons apart before being filled with concrete.

So the specimens at Athens were made, and the tests showed that they gave ample strength. These experiments and others have been described and published in the *Transactions* and elsewhere.

F. Collingwood, M. Am. Soc. C. E., remarked that he would like to add an experience had on the caisson of the Brooklyn Bridge. The conditions were like those described here; at the inner edge the material was very hard, but on the outside it was river mud, compressible although not soft. The caisson was kept level, but when hard bottom was reached at 42½ ft. depth, it was found to be 2 ft. out of position towards the river. The masonry up to high water had been laid abundantly wide, and the error was corrected on the next course of masonry. The movement was totally unexpected, and caused great surprise. Mr. Collingwood.

CORRESPONDENCE.

Mr. Johnson. J. M. JOHNSON, M. Am. Soc. C. E., stated that he had read this paper with a great deal of interest, as it deals with a structure similar to quite a number constructed south of the Ohio River 25 or 30 years ago. The style of foundation then in vogue, as described by the author, was delightfully simple, and only a special Providence, supplemented, however, by a liberal and early use of stone by the railroad company forming loose revetments around the base of each pier, averted disaster.

He agreed with the author's conclusion, that as a service extending over 30 years has developed no appreciable settlement in piers Nos. 2, 3, 4, 5 and 6, the removal of the obstructions due to the old round pier and pier No. 9, with additional revetment at the base of each pier, will render these foundations safe. The additional revetment would in his judgment be advisable, to prevent scouring due to accidental causes. The difference between extreme high and low water of the Tennessee River is about 50 ft., and, at times of flood, immense trees are carried with the current, which, lodging against bridge piers, oftentimes cause a scouring of considerable extent, and that, too, in a remarkably short space of time.

The new superstructure possesses no points of more than ordinary interest or special construction.

The problem of erection, however, was more difficult. The old superstructure was to be removed, the grade raised 4½ ft., and traffic not to be interrupted. The grade was raised gradually on the false-work over the new draw span, long "run-offs" being made at each end, until the maximum of 4½ ft. was attained, with but slight delays to trains.

As time was an important consideration, it was necessary for the old spans to be removed and the erection of new superstructure commenced before the completion of the masonry. He remarked further, that with two sets of men, not serving the same master, both anxious to occupy the track at all times between trains, the question of the proper time allowance to each was no doubt a serious problem to the Chief Engineer of the railway company. The rapid and successful completion of the work sufficiently attests the satisfactory solution of the problem.

Mr. Dunham. H. F. DUNHAM, M. Am. Soc. C. E., called attention to the sketches of Pier No. 7, where timbers are shown bolted to the piles at a depth of 12 or 15 ft. below low water, for the purpose, evidently, of increasing the stability of the group or cluster, and stated that a more usual, if less painstaking, method would be the selection of piles of such diameter and number that when driven they would carry the imposed load from the surface of the water to the bottom of the river without buck-

ling. Then the lateral stability of the temporary structure could be increased, if necessary, by driving piles a few feet away upon either side and drawing the heads at about low water level up to the cluster and bolting them there. He thought it would be interesting to know the actual cost of placing the 4 x 14-in. timbers at that depth below low water and whether it was necessary to employ a diver for any other purpose during the construction of the work. Mr. Dunham.

HUNTER MCDONALD, M. Am. Soc. C. E., in closing the discussion, agreed with Mr. Macdonald that the charge to engineering constitutes too small a percentage of the total cost, but stated that this is not an unusual case, which fact is generally due to causes beyond the control of the engineer in charge, and that the remedy for such a state of affairs can be found only in united action by the various engineering societies. The regulations of the Inter-State Commerce Commission, for the classification of operating expenses, require that salaries of all general officers be charged to one item; otherwise the percentage of engineering in this case would have been larger. Mr. McDonald.

He considered that there was little danger of any trouble from the timbers of the caissons parting on account of buoyancy, as green oak and gum were used exclusively. Besides this, the outside of the caissons was made of vertical timbers, and the alternate layers forming the tops of the caissons were securely bolted to the outside verticals, besides being heavily drift-bolted, each to the other. Vertical planking on the outside of the cribs prevented any parting of the horizontal timbers in the same, by reason of friction or buoyancy.

He further stated that the lateral movement of the caisson of the pivot pier was due to the fact that it began to penetrate on one side first and to the heavy weight of the old pier on the high side, and that until the entire cutting edge began to penetrate, rip-rap would have been ineffectual to prevent lateral movement. The immense quantity which would have been required on account of the adjacent deep holes made this method impracticable.

There was a difference of opinion among those connected with the work as to the effect of lapping the timbers on the stiffness of the caisson. Unless the halving and lapping is done with great precision, he thought the other plan the better, and that the results gained by the former plan do not justify the expense.

He added that the bracing of the piles under the water was considered necessary on account of the likelihood of the gravel scouring from around them and from the fact that steamboats, rafts and other floating bodies might strike the piled piers with great force. The piles driven were selected large ones, and the bracing was an additional precaution. A diver was constantly employed, who, when not working under water, worked above. The expense was comparatively small, although the exact figures cannot be given.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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745.

Vol. XXXIII.—March, 1895.

WIND BRACING IN HIGH BUILDINGS.

By GUY B. WAITE, Assoc. M. Am. Soc. C. E.

READ DECEMBER 19TH, 1894.

WITH DISCUSSION.

There is nothing that seems more important in architectural engineering at the present time than a thorough consideration of the lateral forces to which any of the very high modern buildings may be subjected, and a practical and uniform agreement on the proper system of treating the structures to resist such forces.

On the question of safe construction for positive vertical dead loading there is substantially no disagreement among engineers, and the proper allowances which it is advisable and wise to consider for superimposed vertical loads on floors in such buildings are generally agreed upon, although it is conceded that such loading may not occur more than once in a decade.

After taking into account the nature and the value of the various live forces, which act vertically, horizontally or otherwise, the question naturally presents itself, whether modern buildings are being constructed with the same amount of safety against all of the various

forces mentioned. Certainly no one will question the wisdom of insuring a structure against each of the destructive forces to which it may be subjected, to a degree at least proportionate to their probabilities.

Most designers of high buildings are convinced of the safety of their structures against the ordinary forces which will be brought to bear upon them, but they do not always base their conviction upon scientific reasoning, and therefore it is natural to find a diversity of design proportional to individual opinions.

All engineering designs for high structures, such as towers and chimneys, are made to conform to mechanical principles for the resistance of lateral forces of wind at a pressure of from 25 to 50 lbs. per square foot of exposed windward surface.

While the sizes of building lots in the large cities are not materially changing, the heights of the buildings placed on them are rapidly increasing; so that now numerous buildings extend 250 ft. above curb level, and there are many instances of buildings varying from 23 to 25 ft. in width and from 120 to 175 ft. in height, with the uppermost 100 ft. exposed to wind. Now, from a scientific or engineering standpoint, structures with sections corresponding to the last mentioned should at least have positive resistance, and if the construction is of a character to permit doubts as to its ability to perform a particular function, such doubts should be on the side of safety.

Observers will have noticed examples of buildings having cross-sections corresponding to the latter dimensions given—from 80 to 100 ft. deep and almost identical in character and general planning—where one structure is made to depend upon cross partitions, end walls and the ordinary girder connections, with columns for transferring horizontal forces to foundations, while the other has a distinct system of structural bracing capable of caring for similar forces. It would seem that either one owner was put to unnecessary expense, or that the other has not proper insurance on his structure. In doing justice to the subject, however, it may be said that partitions, walls, etc., as ordinarily constructed, cannot be relied upon to act in conjunction with steel structural work in resisting lateral forces, and, therefore, if the former means can be relied upon in a certain case, there will be a limit beyond which the steel frame must take up such forces, and then we could have examples similar to the one above described,

where one building is within the limits for resistance by partitions, walls, etc., and the other one just beyond that limit. Such a classification can, however, be made properly only after the qualifications of the materials for resisting such forces have been definitely determined, in order to place them on a footing with steel structural work.

Dr. Draper, who is in charge of the Meteorological Bureau at Central Park, recorded a wind pressure of 49½ lbs. per square foot on November 12th, 1883, at 7.30 P.M.; 36½ lbs. per square foot on March 12th, 1888, at 2.15 P.M.; and 44 lbs. per square foot on January 30th, 1894, at 3.15 P.M. On October 10th, 1894, at 4.30 A.M., he recorded 21½ lbs. per square foot from a northeast wind. This was about the time that a building under construction at No. 74 Monroe Street, New York, collapsed. The height of his pressure gauges is only 53 ft. above the surface of the ground, and the protection from surrounding objects necessarily causes pressures much less than would obtain for higher altitudes. The velocity-recording gauges of Sergeant Dunn, on the Equitable Building, give very much greater average records than Dr. Draper's. For instance, during the storm of October 9th and 10th, Dr. Draper recorded an average maximum velocity of 24 miles per hour, while Sergeant Dunn recorded a corresponding velocity for the same wind storm of 48 miles per hour.

The maximum pressures are of very short duration, usually not lasting longer than a second, but sometimes occur at intervals, and thereby tend to set up an oscillation in structures which they are capable of influencing.

After consulting standard authors, reliable data, and prominent engineers, the writer is unable to find any engineer who is willing to assume the responsibility of allowing an average of less than 30 lbs. per square foot horizontal pressure on the exposed windward portion of high buildings.

Since buildings of this class are not ordinarily subjected to horizontal forces other than wind, aggregating an approach to this limit, subsequent discussion will be confined to wind pressures only.

An argument often presented by those who are sanguine about the efficiency of ordinary materials of construction (such as walls, partitions, and general filling-in material) is, that the maximum pressures are simply local gusts of such short duration that their impulses are absorbed in overcoming the inertia of the mass. If this be true in very

high buildings, where thickness of wall is inconsiderable compared with the exposed height, the material of the structure must act as an elastic body, and the wind impulses must be of such short duration that, before the reaction of the elastic mass has taken place, the force of wind has subsided, and the impulse has been exhausted in internal work. Now, if there should be a succession of such impulses and one should come not in harmony with the vibration created in the material, the heavier the mass the more detrimental would its weight become when its oscillation and the wind impulse act together. Furthermore, if the wind impulses are thus dissipated, the prevalent practice of calculating for vertical reactions due to them is erroneous; for no horizontal force on one side of the structure can cause a vertical reaction on the opposite side without being transformed by intervening construction.

The Venetian Building, in Chicago, which is 50 x 108 ft. and 135 ft. above curb, was designed for a total wind pressure of 40 lbs. per square foot. The steel construction was designed to take 70% of the total pressure, and the general mass of the structure was relied upon to take care of the remaining 30 per cent. It may not be unreasonable to assume that a building can be built so that the natural bracing of its general construction and the special bracing of the steel structure may accommodate each other enough to act in conjunction, in which case why cannot 40 or 50% of the entire pressure be taken by the general construction in more favorable cases? To formulate definitely for the various existing conditions, more complete data seem necessary such as—

First.—The maximum amount of wind pressure on good-sized areas at various altitudes, and under all ordinary circumstances, in the especial locality under consideration; also, the length of time during which such maximum pressures act.

Second.—Experiments on the strength of partitions, walls, etc., for resisting vibrations and thrusts under similar conditions to those existing in buildings.

Third.—Experiments to determine to what extent the mass of the structure directly absorbs the impulses of wind pressures.

Under present circumstances, there seems to be no reasonable course of procedure left for the discussion of existing conditions other than to accept authentic data and the judgment of the most experienced authors.

Under this conviction the writer will discuss some structures, the general dimensions of which are in accordance with plans coming under his observation. For the first example let us examine the construction of an ordinary non-fireproof building not classed with high structures. Take a warehouse building (Fig. 1), 23 ft. front and rear, 96 ft. deep, and 85 ft. high, located with an alley-way on one side and a three-story building on the other. Assume that the adjacent building

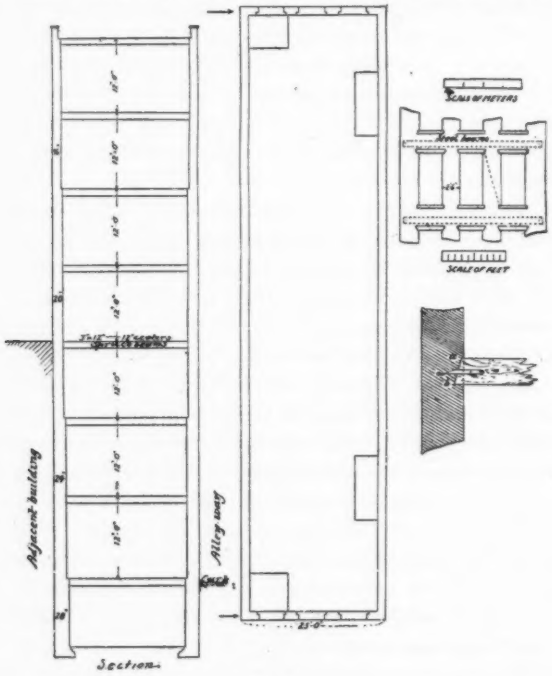

FIG. 1.

is capable of resisting the effects of wind when blowing towards it, and, for brevity, only discuss the effects which wind blowing on the exposed portion above the adjoining building will have on the structure.

The walls are of brick, 28 ins. thick in cellar, 24 ins. in first and second stories, 20 ins. in third, fourth and fifth stories, and 16 ins.

in sixth and seventh stories. First determine what wind pressure the dead weight of walls and floors will safely take care of. When the material in walls is not first class, the center of resistance at each joint is taken within the middle third (to avoid tension on the windward side); the dead weight above each floor level will resist the following wind pressures on the exposed surfaces above the floors:

7th	floor, 5	lbs. for	each	wall = 10	lbs.	
6th	" 3	"	"	= 6	"	
5th	" 2.7	"	"	= 5.4	"	Approximately.
4th	" 2.15	"	"	= 4.3	"	
3d	" 2.2	"	"	= 4.5	"	

As wind ceases at the fourth-floor level and the dead weight increases faster than the other functions as we proceed downward, 4.3 lbs., depending on the joint at the fourth floor level, will be the limiting wind pressure which can be cared for by walls and dead loads taken independently. Now, if the walls are of excellent material and well built, the center of resistance may be taken at one-quarter way from the inner side, which will make a maximum compression on the inside, of about 100 lbs. per square inch, and a maximum tension on the outside of 33 lbs. per square inch at fourth-story floor level; then the total limiting wind pressure which can be cared for by dead weight is 6.5 lbs. per square foot on the entire exposed surface above the adjacent building. This pressure corresponds to a wind velocity of 40 miles per hour. Now, for a pressure exceeding 6.5 lbs. per square foot, the surplus must be carried through the structure to the foundations, and we must look for means in the construction to provide for it.

The maximum shear from wind will occur where the aggregate effect comes on the fourth and lower floors. Then the next problem is to determine what wind pressure the fourth and lower floor systems are capable of transmitting from the horizontal to the vertical.

A wooden floor beam as usually laid in a wall, with beveled edge and not anchored, can exert no resisting moment at its seat; but if its back edge is built up solidly against, and it has a solid seat, and is well anchored in the wall, it can be relied upon to resist something.

In the present case the 3 x 12-in. spruce floor beams, spaced 12 ins. on centers, have every sixth beam anchored. Now, supposing that

such beams have been built into the wall with a view of exerting a resisting moment at their ends, and that they are anchored with straps with net sections of $1\frac{1}{2} \times \frac{3}{8}$ ins., and fastened on center line of beams with two $\frac{3}{8}$-in. spikes, we can allow for such resisting moment 6 000 in.-lbs. Such moment will resist a wind force on a wall 50 ft. high by 6 ft. wide (when considering the wall rigidly held at each floor level) of about $\frac{3}{8}$ lb. per square foot at each end of the beam, or a total of $\frac{3}{4}$ lb. per square foot. Had all beams been thoroughly anchored, they would have sustained a wind pressure of $4\frac{1}{2}$ lbs. per square foot. If each beam had been anchored by wall plates at both top and bottom, they would have resisted a total of 9 lbs. per square foot wind pressure on exposed wall. Thus far we have found a total resistance for structure as follows: walls, 6.5 lbs.; floor beams, .75 lb. = 7.25 lbs. This is equivalent to a velocity of 42.5 miles per hour.

This velocity will cover that of ordinary winds, but in a violent storm the velocity will reach 70 miles per hour on a building of this height located in New York City, with surroundings as described. That is equivalent to a total wind pressure of 20 lbs. per square foot, which should be provided for in a structure of this kind. This leaves 20 lbs. — 7.25 = 12.75 lbs. to be provided for in some part of the construction not yet considered. If floors are well constructed throughout, they, with the walls, will form a truss sufficient to transmit such a horizontal pressure to the ends of the building, where, if sufficient provision has been made in the fronts, it can be conducted safely to the foundations. We will assume in this case that fronts have three windows in each story, with piers 30 ins. wide between them; in the panels above and below the windows are two steel beams sufficient for all emergencies anchored into the wall at each end. Now, the girders over and under each set of windows, with the filled-in brick work between them, form a truss which must transmit the total shear at the fourth-floor level of 30 600 lbs. The girders by confining the brick work act in the capacity of ties, and the piers as diagonals in transforming horizontal pressures into vertical ones. We can assume that the stress is equally divided between the four piers, then we will have a resultant diagonal strain on each of 25 000 lbs. The pier being 20 ins. thick, and allowing 100 lbs. per square inch pressure, an active diagonal section of brick of 20 x $12\frac{1}{2}$ ins. will be required. Such piers

should properly be made of good hard brick laid in cement, to insure an efficient diagonal. The actual bearing area on the diagonal edges of piers may be quite small if of good material.

As the dead weight in the front greatly exceeds the vertical reaction, the latter will not be discussed here.

A practical illustration of the partial transmission of horizontal pressures to end walls by the floor system, and the resistance of such end walls to horizontal forces, can be seen in the buildings located at Nos. 250 and 251 South Street, New York City. The buildings are each about 24 ft. 6 ins. front and about 150 ft. deep, extending through from South to Water Street. There is a small break in the party wall at about 90 ft. from South Street of 1 ft., which slightly stiffens it.

No. 250 South Street is six stories high, or 65 ft. above curb. No. 251 South Street is five stories, or about 50 ft. high. Floor beams are 4 x 12 ins. spaced 16 ins. on centers, and each building has a row of columns and 10 x 12-in. girders. Columns are 8 ins. diameter, cast iron, for three lower tiers, and 8 x 8-in. yellow pine above, spaced 10 ft. on centers. The walls are 24-in. stone foundation, and 16-in. brick upper walls.

No. 251 South Street has only a one-story blacksmith shop on the east side, and is therefore exposed to northeasterly winds on the four upper stories. No. 250 South Street has only the upper, or sixth, story exposed on the easterly side.

No. 251 South Street is leaning westerly 5 ins. at the front and increasing in extent to about 30 ins. at central part. The foundation walls are in perfect condition, and no settlement of either has taken place. The easterly exposed wall has an appearance of being "scooped out," owing to the bottom part remaining nearly in line, while the upper part has yielded most at the center, gradually diminishing at the ends where it receives its support.

No. 250 South Street has yielded and leans similarly to No. 251, as also the three or four adjoining westerly buildings, but to a lesser extent, and it seems to be mainly due to the westerly buildings that the ones described are held in place.

Since the above was written, the sad details of the fatal wrecking of the new warehouse building in course of construction at No. 74 Madison Street adds another to the list of buildings collapsing by wind pressure.

The building was 20 ft. front, 92 ft. deep and seven stories, or 75 ft., high above curb level. The bearing walls were 24-in. brick foundation, 20 ins. through first and second stories, and 16 ins. thick above. The floor beams were spruce, 3 x 12 ins., spaced 16 ins. on centers. During the storm which wrecked the building, the maximum velocity recorded at the United States Weather Bureau on the Equitable Building for one hour was 48 miles, which would mean an average pressure of about 10 lbs. per square foot.

At the Meteorological Observatory in Central Park the maximum impulse (which lasted but one second) occurred at 4.30 A. M., and was 21¾ lbs.

The wind evidently caught the unfinished building nearly broad side, but at a sufficient angle to throw a considerable pressure against the leeward wall through the unfinished open front. The front portion of the building was completely thrown over down to the foundation, crushing the adjoining two-story frame structure, while the rear portion of the building was held up and stood after the wreck by means of the completed rear wall.

Not being able to determine the exact conditions at the time of the accident, the wind pressure which was required to collapse the structure cannot be determined.

For a second example, we will refer to a skeleton-constructed 17-story fireproof office building (Fig. 2), located on the corner of two streets with 32 ft. front, 100 ft. deep, and 194 ft. high above curb level. The long side not facing the street is protected from wind pressure by an adjacent building for 50 ft. above curb; and in order to be liberal in our considerations we will assume the adjacent building capable of resisting the thrust from wind pressure on the structure under discussion when acting against it. Then the maximum effects of wind on columns, girders and connections will be when it is blowing at right angles to the portion of structure over the adjacent building.

The maximum strain on girders, bracings and the connections, also the bending in columns, will occur in the floor system at the level of the adjacent roof and all tiers below it.

The maximum vertical reactions in columns and foundation will occur, of course, in the lowest tier of columns.

For the transmission of stress from horizontal to vertical members, we will assume that form of bracing which is readily introduced into

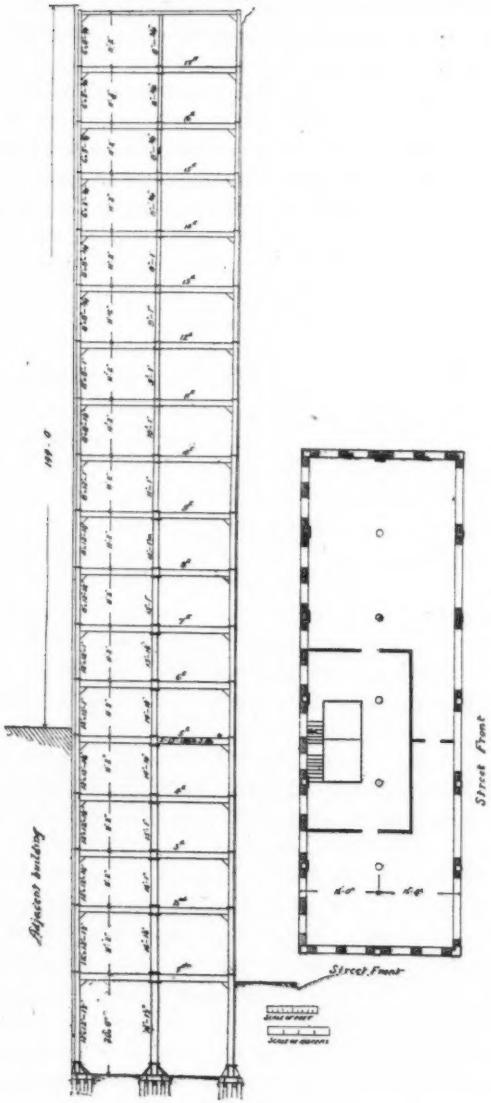

FIG. 2.

this class of buildings, consisting of knee bracing located at the juncture of the floor system with the columns. The dead weight of the walls and floors is conducted by means of supporting girders to the columns, and the amount of wind pressure which it will directly counteract depends largely upon the kind of column connection.

Suppose the 12 x 12-in. columns which connect at the fifth tier of beams have strong 3½-in. flanges, making the total width of base 19 ins., and allowing the center of resistance to come within 2½ ins. of edge of flange, the total resisting moments from the dead weight at the three rows of columns will counteract an aggregate wind pressure on the exposed surface of about 4 lbs. per square foot, which corresponds to a velocity of 31 miles per hour.

For the benefit of those who are sanguine about the efficiency of partitions and cross walls for resisting horizontal forces, assume that they have been especially built with that purpose in view. The partitions will seldom be located concentrically with reference to the center line of wind pressures, and their location will often vary in different stories, but the floor system with arches completed is capable of distributing the pressures, and their eccentricity need only be taken into consideration when it is so great as to permit danger of torsional movement.

Suppose the partitions are of 4-in. terra-cotta blocks laid in Portland cement and have doorways as shown in plans (the floor beams are assumed to be able to withstand the stress imposed by partitions when acting as braces without appreciable deflection), then we will have in principle a system of trusses with partitions acting as struts. On account of the tendency to side flexure, the allowable unit stress must be taken very small. Allowing 25 lbs. per square inch for partitions laid in cement 11 ft. high, the aggregate capacity of those in fourth story will resist a horizontal force of 10 000 lbs.

End walls have five mullions 20 x 20 ins., 8 ft. high, which will resist a horizontal force of 10 000 lbs. at each end wall at 100 lbs. per square inch, mean compression. Hence the capacity of partitions and end walls in fourth story aggregate a total of 30 000 lbs. horizontal force, or an average of a trifle over 2 lbs. per square foot of exposed surface. The total resistance thus far discovered provides for a wind pressure of about 6 lbs. per square foot, or a velocity of 38.5 miles per hour. Therefore, to provide for a velocity of 87 miles per hour, or a pressure

of 30 lbs. per square foot, the bracing provided in the steel structure must be capable of taking 24 lbs. horizontal pressure per square foot. The aggregate shear for 24 lbs. pressure at fifth tier of beams for one bay is 55 500 lbs. This will be equally divided among the three braced connections of columns and girders, so that the horizontal force to be transformed by each will be 18 500 lbs. In view of the uncertainty of cast-iron columns and their imperfect connections for resisting transverse loads, it is but fair to consider them as hinged at each

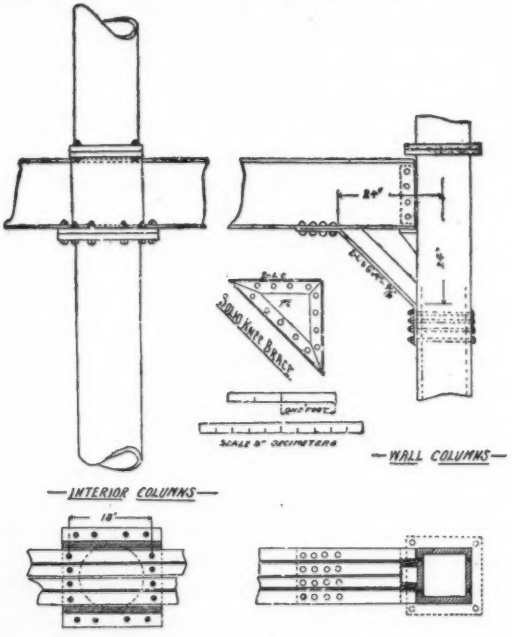

FIG. 3.

story in determining the stresses for knee braces, bending in columns, etc. By referring to detail it will be seen that the 18 500-lb. shear at the foot of fourth-story columns will cause a horizontal stress of 101 750 lbs. at foot of knee brace and an end reaction at girder connection of 83 250 lbs. The stiffener of the knee brace being at 45°, the stress on it will be 143 500 lbs. The bending moment from the braces on the floor girders will be 138 300 ft.-lbs.

The moment of resistance of the 12 × 12 × 1¼-in. cast-iron column being 175, the extreme fiber strain caused by the transverse load at foot of knee will be 11 400 lbs. per square inch at fourth story, and at all lower tiers having 12 × 12 × 1¼-in. columns. The direct dead load over second story is 470 000 lbs. on the 12 × 12 × 1¼-in. columns, and the aggregate live load at an average of 50 lbs. per square foot is 96 500 more. The vertical reaction due to 26 lbs. wind pressure will be 105 000 lbs.

Taking the dead load and the vertical wind reaction together on the leeward column in second story it will be strained by a direct load to 10 500 lbs. per square inch, and in fourth story to 9 000 lbs. per square inch. At the foot of brace, where the transverse load causes an extreme fiber strain of 11 400 lbs. per square inch, the metal will be strained to 21 900 lbs. per square inch compression in second story and 20 400 lbs. in fourth story, without considering the live loads which may be on the floors.

This amount is in excess of allowable stress on cast iron, and under the conditions it is believed to be too much for safety.

The connection of the top of column to the floor girder must be able to resist a horizontal force of 83 250 lbs., and the connections at foot of columns themselves must resist a shear of 18 500 lbs., not considering friction. The knee brace should have a section of two 6 × 4 × ⅝-in. steel angles riveted together, and with sufficient bolting to the columns and to the girders to take a shear of 101 750 lbs. The vertical reaction at seat of girder due to brace will be 89 000 lbs., and connections must be able to take the resultant of this amount and the 83 250 lbs. horizontal reaction with no load on floors. The same general conditions exist at the windward side as at the leeward, except that all moments are opposite in direction. With steel columns and rigid connections, making them continuous, and the knee brace with a substantial web sufficiently connected with column and girder to make all act as one piece, the point of contraflexure will be half way between lower column connection and foot of brace. The maximum bending in column will occur at foot of brace and at column connection, and will be 999 000 in.-lbs. If the column should have a moment of resistance of 175 (same as cast-iron column), the maximum fiber strain would be about 6 000 lbs. per square inch, while the strain on knee brace would be reduced practically one-half.

Considering the interior column hinged at base and the cross-girder connected to its top, as shown in detail (Fig. 3), the maximum bending from wind pressure in the column will be 2 000 000 in.-lbs.

The moment of resistance of the 14-in. diameter column, 1¼-in. metal, being 147, the extreme fiber strain will be 13 600 lbs. per square inch.

The dead load coming on the fourth-story column is 250 000 lbs., and the direct stress therefrom is 5 000 lbs. per square inch, or, assuming an average live load of 50 lbs. per square foot floor surface (in addition to dead load), the direct stress will be 8 000 lbs. per square inch.

This will cause a maximum compression of 18 600 lbs. per square inch in the first case, or 21 600 lbs. in the latter case.

The bending moment at the top of the column must be resisted by the connections of the girder to it; supposing the centers of girder bolts on the opposite sides of column are 18 ins. apart, the vertical reaction at each connection to girder flange will be 111 000 lbs., for which force the bolting must be made sufficient.

Buildings of this description should be provided with steel columns, and it will generally be found economical to increase the section where maximum bending occurs to take up the transverse loading.

In conclusion, the writer begs to refer to an instance of the yielding of a skeleton-constructed building to wind pressure. The building was 23 ft. front, about 85 ft. deep, and 125 ft. high above curb. It was protected from wind by buildings 60 ft. high on either side.

The structure was designed to resist wind pressures by means of front and rear walls and the special connections of cross-girders with columns. The constructor evidently did not consider the importance of these details, ignored them entirely, and, instead of utilizing the entire depth of girder, he gouged holes for fastening it to columns in the middle part of the girder webs. When the walls were up (except the top story) and the floor arches were yet to be put in the three upper floors, the building, during a wind storm, moved westerly from 8 to 10 ins., beginning at the roof of the adjoining westerly building and increasing to the top. The reason is evident: the building, under the force of the wind, had to move until the joints came to a firm bearing, and there it remained stationary.

The building was righted and made secure by elaborate knee bracing, made sufficient to take the horizontal force independently of the original connections.

The writer would call attention to the necessity of carefully executing the details of construction in structures of this character. Here the safety of the building will depend as much on the ability of the constructor as of the designer. Without the judgment to discriminate between vital parts and secondary ones, unnecessary work may be done in one instance or the structure may be endangered in another. The aim of the writer has been to present conditions rather than exact data, in order to encourage discussion which shall yield practical results.

DISCUSSION.

Mr. Clarke. T. C. CLARKE, M. Am. Soc. C. E., desired to know if the author disapproved of knee bracing, as he had said that he did not mean to advise any kind of bracing.

He wished to call attention to the fact of the presence of Mr. George B. Post, who had strongly expressed his opinion at the last meeting of the Institute of Architects as to the great danger of corrosion of iron and steel columns as applied next to outer walls of buildings, and to ask him if he is still of the same opinion.

Mr. Purdy. C. T. PURDY, M. Am. Soc. C. E., stated that the subject is one of peculiar difficulty on all sides, because the forces to be resisted are not agreed upon. The resistance of walls, either crosswise or lengthwise, is not definitely determined, and the same is true of partitions of all kinds. If more resistance is required than these afford, it must be provided in the metal frame, but so long as these first factors are uncertain, the work required of the metal bracing is correspondingly indefinite. The position of the bracing in the building and the method of its construction are also generally troublesome problems, and the strains are not always easy to determine. The stiffness of buildings is, of course, as important in small buildings as in large ones, but he could speak only of the latter, having had little experience with the former. It seemed to him that every pound of weight in a structure acts to some extent in resisting lateral movement, and that the equilibrium of the different parts of the building must be disturbed before there is any need of bracing. A well-made wall is not easily overturned crosswise, while lengthwise it acts as a brace. The conditions are not the same in any two buildings, and if any principles that will cover general conditions can be laid down, a long step forward will have been taken. Metal bracing must always be constructed in such a way that it will not interfere with either the appearance of the building or its usefulness. It is exceedingly difficult to do this in most buildings, and it is

very important that metal bracing should be tight in every connection, for if there is any play or movement possible between members, it cannot be very efficient. At various times he had found it impossible to use diagonal rods, and had substituted heavy plate work in the shape of portals. This construction has a great advantage in its riveted connections at all points, and it does not matter from which direction the wind comes—all of the metal is always in service. Mr. Purdy.

He further remarked that partitions are often omitted in steel-constructed buildings, and in his practice not much dependence is placed on their resistance to wind in regular office buildings. If they are omitted in one floor it is nearly as bad as if they were omitted in several.

There is a method of calculating the resistance due to the weight of the material in structures, but in most steel buildings it is not a great factor. The thick walls required by the New York building law add much to the lateral strength of buildings here. Any system of metal bracing connecting two columns of a building makes the columns act like the chords of a truss, one in tension and one in compression. When this tension stress becomes so great that it reduces the dead load on the column to zero, the limit of the efficiency of that line of bracing is reached unless the whole is anchored to the earth and the joints in the columns are spliced for tension, all of which is impracticable. In most large buildings there are some points where it is possible to use bracing of some kind. In his practice these places are searched out, and the bracing made as strong as the dead column loads will permit. If all the resistance that seems to be required is secured, it is satisfactory; otherwise, other places are sought where the building may be further strengthened, or, if necessary, some further opportunity is forced. The weight of the building is always taken into account, and its resisting effect actually calculated as nearly as possible. This is a tedious operation, however, which would take some time to explain, and is, moreover, largely a question of judgment. He always tried to be sure that there was lateral strength enough in the steel construction to resist any ordinary wind, and the fewer partitions there are in the building, the more particular he was about it. In many buildings he alternated the columns in double lengths, and relied upon strong connections for stiffness. He called attention to the fact that floors are good conductors of horizontal forces, and also stated that he generally required eight rivets in each end of each beam connecting to columns in such construction.

GEORGE A. JUST, M. Am. Soc. C. E., observed that it was gratifying that the author had brought this subject before the Society again, as his position in the Department of Buildings of this city afforded a wider field for observation and opportunities for criticism, by virtue of the many plans placed before him daily, than that of any engineer Mr. Just.

Mr. Just. engaged in private practice. He is the first (recognizing the fact that buildings are being erected without accident all around us every day without any *intended* provision against the action of the wind) to attempt to give to each of the elements entering into the construction of the same, although not employed with this end in view, a proper value of resistance to the force of wind to which the structure might be subjected.

He pointed out that the author had taken as a first illustration a building practically the same as the one which collapsed in Monroe Street, New York, during the storm of October 10th, 1894; and had found in the walls and beams alone a resistance value of $7\frac{1}{4}$ lbs., equivalent to a velocity of $42\frac{1}{2}$ miles per hour, and, further, that with beam girders and brick piers of certain size in the fronts, they could take up $12\frac{3}{4}$ lbs. additional pressure, or a total of 20 lbs., equivalent to a velocity of 70 miles per hour, which is about as great as any yet recorded here. It is known that buildings of this type with no special provision to resist wind have been erected and are now standing in this and perhaps other cities, and that the collapse of the Monroe Street building would in all probability not have occurred if the front and the window sashes had been in place at the time of the storm. The fact that the building did collapse is, of course, proof that proper provision had not been made at that stage of the construction when this particular storm prevailed, and he had, of course, no intention to encourage such a system; yet the case afforded a very striking example of either the wind-resisting qualities inherent in this type, built entirely without intention to provide against this force, or that the total resultant effect of the wind, even in high storms, is not as great as ordinarily assumed.

The fact was that the building collapsed from the cumulative effect of conditions all prevailing at that time. The plans had been approved by the Department of Buildings and the materials used in construction were as good as are generally used for this class of structures. However, Monroe Street and other parallel streets in this section of the city, run almost due east and west, hence the front and rear walls had this direction, while the gables ran nearly due north and south. The wind blew from the most unfortunate quarter for a building so located, coming from the northeast, and therefore striking not only the easterly gable at a slant, but entering as well by its open windows and by the front not yet fully erected, thus exerting its pressure over the outside of the easterly and the inside of the westerly gable wall at the same time. The rainfall had been heavy (nearly $1\frac{3}{4}$ ins. in 13 hours), and had soaked the green walls. The wind blew at the rate of 37 miles an hour at three o'clock A. M., about the time of the accident, and reached a maximum of 48 miles, and for a short time 60 miles per hour. The storm can well be called extraordinary for this section, since its

effects were widespread. This storm also caused the collapse of the guide frame of the gas-holder of the East River Gas Company, an open iron-work structure, 198 ft. in diameter and 150 ft. high (the erectors having failed to put in place the diagonal bracing as the work progressed), the chimney of the New York Terra Cotta Company and the walls of a fire engine-house in course of construction, all at Ravenswood. The house of Captain Merritt at Whitestone, L. I., was unroofed. Vessels were torn from their moorings, and much damage was done to shipping all along the coast and Sound, the havoc caused extending as far north as Halifax. A two-story frame house in Brooklyn, N. Y., in process of erection was destroyed, and plate-glass show windows were blown in. At Long Branch, N. J., a portion of the West End Hotel roof was carried away, and summer-houses, greenhouses and barns were demolished. It would also appear that a wind-storm of certain intensity is sometimes not so destructive as a rain-storm combined with a wind of less intensity. The wind-storm of March 28th, 1895, broke the U. S. Signal Service record of 72 miles in New York City, which had stood for 29 years. It blew at the rate of 75 miles per hour for two minutes, and an average of 55 miles per hour during 12 hours. Yet this storm was not nearly so destructive as the mixed storm of October 10th, 1894. The only serious accidents reported were the foundering of a tug in the Upper Bay, the falling of a frame ventilator 5 ft. square and 60 ft. high, from which the surrounding scaffolding had not yet been removed and the collapse of a three-story frame building, just roofed, both in Newark, N. J. In comparing these storms it is necessary, however, to point out that in the rain-storm of October 10th, 1894, the wind was from the northeast, and measured by U. S. Signal Service instruments placed 200 ft. above the ground, while the wind of March 28th, 1895, was from the northwest, and was measured at an altitude of 326 ft.

Mr. Just.

He called attention to the fact that in the second or "three-column" example, the author exhibits a structure which, while not actually built, was at least seriously proposed, and gives a resistance value of 4 lbs. to the dead weight, 2 lbs. to poor partitions and a "very open" front, and looks for bracing which shall supply 24 lbs. additional, which, when supplied, induces unit strains in the cast-iron columns incompatible with safety. The author's last example, easily recognized as the so-called "Diamond Exchange" on Maiden Lane, was designed and erected with cast-iron columns. In this case, however, the special details designed for the connections were ignored by the contractor, hence distortion of the frame, which was, however, readily pulled into plumb again, without damage to the brickwork and other parts of the building which were not at the time sufficiently advanced.

It seemed to him that the last two examples only emphasize what has already been pointed out in the earlier discussion of this subject,*

* *Transactions*, Vol. XXVII, p. 221.

Mr. Just. viz., that cast-iron columns should not be used for buildings of narrow base and extreme height, because of the bending induced by necessary bracing, and because the old architectural iron-workers have no sufficient appreciation of the essential details of such constructions and can hardly be made to execute them effectively, even when prescribed.

He thought this whole subject for convenience could be really considered under three distinct heads, viz.:

First.—A more exact determination of the maximum pressure of wind, a better expression for which would be "total resultant effect of wind."

Second.—A determination of the value of the wind-resisting elements in existing buildings, not built with such a view.

Third.—The adoption of more uniform and rational methods in calculation and hence design of modern structures in which the effect of wind is recognized and taken into consideration.

This last, he said, is now receiving more attention from designers of the more recent and better examples of high construction, but the first seems to be as indeterminate as ever. One is first struck by the extreme variations in the records of different observers in the same locality. Thus, for the storm of October 10th, 1894, and for the greatest storm yet recorded in New York City, Dr. Draper, of the Arsenal in Central Park, and Elias Dunn, of the U. S. Signal Service, in the Equitable Building, report the figures in the table on page 209.

These variations are hardly to be explained by the difference in location and elevation of the instruments of the two observers.

In the case of large buildings or on other exposed surfaces of great area, the wind does not act in the manner assumed for the purposes of calculation. The pressure is not uniform, it is seldom normal, and, generally, is not simply progressive in motion, but also rotary.

Of course, if the "total resultant effect" of a wind were well known, an equivalent uniform pressure applied to so-called panel points could still be used, but it would be probably less than that now usually assumed. Hurricanes are excluded from consideration, but the fact that the walls of partially completed buildings do not fall more frequently under the action of ordinary winds is perhaps explained because they are seldom normal to the plane of the wall, and hence less effective than those recorded by machines which receive the pressure normally as well as by the fact that the higher pressures recorded are local and not of the same intensity when spread over larger areas.

He further stated that the relation of pressure to the slope the surfaces of a body present to the direction of wind has recently been determined, perhaps more accurately than ever before, by the experiments of Mr. J. Irminger, of the Danish Society of Engineers, who used velocities of about 16 and 33 miles per hour, and found, for instance, that the

TABLE OF DATA RELATING TO STORM OF OCTOBER 10TH, 1894.

INQUIRY.	DRAPER.	DUNN.
Velocity of wind, 3 to 4 A.M., October 10th, 1894	19 miles per hour.	37 miles per hour.
Maximum velocity in this storm	20 miles per hour.	48 miles per hour, for five-minute intervals; 60 miles per hour, extreme recorded.
Equivalent pressure per square foot	21½ lbs.	11½ lbs. for 48 miles. 18 " " 60 "
Kind of machine registering velocity	Draper's self-recording.	Robinson anemometer cups.
Kind of machine registering pressure	Draper's self-recording.	By table, calculated from Smeaton's formula— $p = \frac{V^2}{200}$
Height of velocity. Machine above ground	53 ft.	200 ft.
Height of pressure. Machine above ground	53 ft.	200 ft.
Rainfall	1.67 ins.	1.71 ins.
Number of hours in which same was measured	12 hours.	13 hours.
Quarter from which wind blew in this storm	E. N. E.	N. E.
Greatest velocity ever recorded here	41 miles per hour.	72 miles per hour, on a five-minute interval basis.*
Greatest rainfall ever recorded here	8.28 ins., September 23d, 1882.	1.6 ins. in one hour 6.17 " " 24 hours.
Number of hours in which this was measured	19 hours 30 minutes.	One hour and 24 hours as above.

* This record, made in 1876, was exceeded at 11 A.M., March 28th, 1895, when the wind blew from the northwest at the rate of 75 miles per hour for two minutes, measured by the instruments of the U. S. Signal Service in the Manhattan Life Building, 326 ft. above the ground.

pressure on a lozenge-shaped body presenting two of its surfaces at a slope of 30° to the wind (*i. e.*, the two surfaces presented forming an angle of 60° between themselves), terminating at a distance that made the vertical between them "S" units high, was only 25% of that exerted against a plane of equal height placed normal to the wind. He found also that in the former case the total pressure exerted was made up of 18% of pressure on the windward and 82% of suction on the leeward side, while in the latter case pressure and suction were nearly equal, viz., 45% on the windward and 55% on the leeward side. Of course, these experiments are not directly applicable to the case of buildings, since in the experiments the body experimented upon was entirely immersed in the pressure-producing medium, which is not generally the case in large structures.

In conclusion he remarked that Capt. W. H. Bixby, M. Am. Soc. C. E., had also made a comprehensive compilation of the literature on the subject of wind pressure, which was recently published by the Government in a report to the War Department.

Mr. Post. Mr. GEORGE B. POST, remarked that he was not prepared to speak scientifically on the subject of wind pressure in high buildings, although he had read the paper with a great deal of interest. It had been his fortune to put up a great many large structures, but, being very conservative, he had had a fear of various disturbing elements in cage construction, and had preferred to build the cage inside of the walls and anchor the walls to it. In almost every case in large buildings which he had erected, the rear walls of the building opposed to the wind forces were recessed and broken in such a way that the horizontal strain of wind from the exposed fronts would be carried by the floors from the front to the rear walls where it would be safely taken up by the stability of the latter.

He called attention to the Havemeyer Building, 114 ft. long, 175 ft. high, with a westerly front that is entirely exposed to the winds. The building is about 47 ft. wide for 50 or 60 ft. in depth from each end, and at the center of the rear wall there is a round structure for elevators, making a tower of great stability. There are two continuous lines of diagonal cross-bracing at a distance of a little over a quarter of the length of the building from each of the extreme ends, put in for the purpose of taking up any accidental wind strains. The columns stand free on the inside of the walls, and the wind bracing itself was, perhaps, hardly necessary.

He desired to speak of one difficulty which may possibly arise from making a steel frame very rigid by knee bracing. In spite of the greatest care which can be taken in design, it is almost impossible to be absolutely sure in all cases of equal settlement. Unequal settlement has occurred in some cases where the greatest care has been taken, and in such examples unexpected and severe strains might be brought on almost any part of the steel cage, which might have serious results. In case of a rigid steel-cage construction, if the settlement in the wall at the side of an adjacent building should be much greater than on the other side, the tendency would be to throw the whole burden of sustaining the pressure upon the adjoining building, or to make the building itself overhang the adjoining lot. If a building 175 or 200 ft. high, with a steel cage enclosed by a few inches of outside brickwork, should overhang a lot and an order of the court was procured to cut off that overhang, he would like to inquire what would be done with it and how it could be jacked up again?

In the old system of construction, where the walls are calculated to take care of themselves, with a reasonable amount of bracing from the floors, if one wall settled a little more than the other, the floors got out of level, yet the walls went down straight. He said that he was watching with considerable interest to see what would be the effect of settlement on some of our modern steel-cage construction.

Mr. Post.

In reply to Mr. Clarke he stated that his remarks on the occasion spoken of were falsely reported, but he had heard a great deal of it since and, if the report was correct, he said what he did not intend to say.

He thought that the great danger in our present system of construction is from corrosion, unless the greatest possible care is taken during the course of construction to protect the metal from subsequent alternate wetting and drying. He observed that steel-cage construction put up without this proper care is dangerous, and believed that the time would come when there would be failures among these buildings from corrosion of important bearing points. He had had long experience with both cast and wrought iron in connection with masonry; but a very small experience, of course, with steel, as steel is a matter of recent introduction in building construction. The first tier of beams of the New York Times Building, when he altered it, had been in position for at least 35 or 36 years. They were encased, not in plaster or in light arches, but in the heaviest kind of solid brickwork. With the exception of the lower flanges of the beams, they were vertically encased in brickwork; yet many of the beams were discovered to be so entirely destroyed by corrosion that the flange could be broken with the fingers. He also had had much experience in observing the way in which moisture will go through brick walls. He had built a great many tall buildings, and some not so high and with very thick exposed walls. In all storms the moisture penetrated through the northeast walls of the Mills Building, nearly 4 ft. in thickness, and the water ran down on the inside face, although the wall was laid up with hard North River brick with Norton's cement mortar, and was as good a piece of masonry as he had ever seen, and it was not until this outside wall was painted that the water ceased to leak through at the particular face which was exposed to the storm.

He further said that in the remarks which he made at the meeting of the Institute of Architects upon the danger of this form of construction, he had intended rather to enforce his views of the necessity of the most careful protection from corrosion and to state his firm conviction that many buildings had been put up with such recklessness that they would suffer in time from corrosion; not that it was impossible to build steel-cage buildings with safety.

He mentioned that these cases of corrosion were with wrought iron, but he believed the danger would be much less with structural steel. It had been the pride of his profession that its work should endure for ages; the great architectural works of the world have stood for thousands and thousands of years, and he looked with considerable disfavor upon any system of construction which might be relatively short lived. He proposed to build steel-cage construction, and also to use the utmost ingenuity of which he was capable, to protect the iron structure, so far as possible, from the effect of alternate wetting and drying.

Mr. Emery. CHARLES E. EMERY, M. Am. Soc. C. E., called attention to one important difficulty in this class of construction, viz., that of obtaining a proper bearing of the ends of the columns, one upon another. He said that if a construction going on in the city of New York were closely watched it would be seen that if a column did not show plumb, a helper would simply get a piece of plate iron and push it in the joint until the desired result was secured. He had seen this done, not only between the faced joints of cast-iron columns, but between the flanges of very heavy steel plate columns used in the best construction. In one building a large supply of plate iron "shims" of different thicknesses, cut to the shape of one side of the column, was kept on hand, and, when one was inserted, a few slips and a little putty and paint made the whole joint look solid. His observations were made from office windows of adjoining buildings, and show, therefore, that this method of treatment is adopted even for the lower stories of important buildings where the loads are necessarily quite heavy. He had been informed, also, that in some cases the workmen do not take the trouble to get "shims" of plate iron, but use common nails in the joints to make the columns vertical. He concluded that at the best when one side of an important joint like this is cobbled up with "shims" or "Dutchmen," the bearing is really on two lines, and if it be necessary to add a second piece to make a correction in the other direction, the bearing may be on a few points only. He added that such a construction necessarily frustrates all attempts at calculation of unit strains and would seem to make superfluous all refined deductions based on the differences in formulas for columns with square and rounded ends. He realized that there would be difficulty at times in correcting such an evil, but he had successfully accomplished it in the construction of the New York Steam Company's building where the interior loads were carried by heavy cast-iron columns carefully faced and centered by socket joints. He said that though the joint on the base had been leveled up, the top of the first column set did not prove to be level, and the cause was found in the fact that the column had been faced by the contractor in a lathe with two independent revolving heads, instead of in a gun lathe with the column itself revolving. The lathe heads were, however, put in line, and the difficulty finally overcome without the use of "shims."

Mr. Brinckerhoff. H. W. BRINCKERHOFF, M. Am. Soc. C. E., stated that about 1880, when the Brooklyn Bridge was under construction, an anemometer, somewhat similar in design to that mentioned by Mr. Leverich (see page 227), was set up in the center of the river span on the cradle cables. He was then engaged in the drawing room of the bridge, and he thought the design of the anemometer was suggested by George W. McNulty, M. Am. Soc. C. E., and that the instrument was made and set up by A. V. Abbott, Jun. Am. Soc. C. E., both then engaged on the

bridge. It was at first arranged to be self-registering, but as maximum pressures only were looked for, the self-registering device was used but a short time, and after that it was only noted how far the plate was pushed back by the wind. The plate had an area of about 2 sq. ft. He believed it to have been, perhaps, the first anemometer made to measure and record wind pressures directly without regard to velocity. Mr. Brinckerhoff.

Referring to wind bracing, he said that in the new Manhattan Life Building, it does not extend below the top of the adjoining buildings, leaving the structure at that point, apparently dependent upon its neighbors for support. The building to the north of it is a very slight affair and the building to the south is not a very heavy one, and with the large hall on its lower floor would probably not offer much resistance to horizontal stresses.

He stated that the large chimney of the New York Steam Company's boiler house, Station B, was topped out 20 ins. thick solely for stability, and is believed to be safe for a wind pressure of 40 lbs. per square foot.

F. W. Skinner, M. Am. Soc. C. E., remarked in reference to the Manhattan Building, that he had the privilege of seeing nearly all the working drawings and specifications, and that in most respects they are more carefully and more completely designed than is customary in steel buildings. There are two separate sets of wind bracings. The tower has compression members and diagonal members which in appearance are entirely sufficient to take care of all wind pressure on the tower and transmit it to a solid floor in the roof or eighteenth story of the building, which is made of corrugated floor plates filled solid with concrete, to distribute the strains to several vertical panels of what he would call the second system of bracing, which takes the wind pressure down 150 ft. towards the foundations, which are more than 100 ft. lower yet. There are quite a number of panels between columns, he thought as many as six, in which all the columns have knee braces at each floor beam intersection. These knee braces consist of 3 x 3 x ½ in. angle iron in pairs, with three rivets through each end of each angle, and they are placed at connections down to the sixth story. He considered that it is but fair to add that the sixth story is the place where most of the columns change from riveted sections of rolled steel, having a more or less eye-beam shape, to cast-iron columns—the connection of the knee bracing to cast-metal braces in some degree accounts for the omission of the brackets below the riveted work, but of course it does seem strange to see, in all but one panel, the knee bracing stop off in mid air. Mr. Skinner.

There is one point regarding wind pressures, and particularly the contradictory statements and records of conflicting pressures and velocities, which he had never seen brought forward, and he would like to raise the question for the sake of discussion. There is a very consider-

Mr. Skinner.

able difference of pressure due to the different densities of air between the limits of a very hot, dry air, and a very cold, saturated atmosphere. His attention had been called in this direction a little more than a year ago when Richmond County was visited by three storms of great intensity, each one said in turn to be the severest storm in many years. In one of these storms the fact was noted that it was a comparatively dry storm, and that very little damage was done to the trees; the following storm was with a wind of similar direction and intensity, but under different conditions of moisture, with the atmosphere near the limit of saturation, and great damage was done to the trees. Before a scientific society of which he was a member, a paper was presented by an engineer who went into the matter in detail and advanced the opinion that different wind pressures result usually from the densities of the atmosphere.

He further stated that it is interesting to note the steps that have been taken in the most recently constructed buildings to provide for wind bracing, and instanced one on Nassau Street where the wind bracing is provided for in many of the columns by solid plate bracket knees braced 2 ft. on the side with two angles on each of the three sides of the web; this building is not particularly exposed, being pretty well surrounded by others, and this is the only provision made for wind bracing. In the Masonic Temple at Chicago, which has been built perhaps four or five years, there are two sets of sway-bracing rods; the vertical diagonal rods of one system extending but one story, making the columns of one story and the two successive floor beams form the struts for each panel of wind truss. The panels of the truss of the other system cover two stories, with the screw-rod diagonals intersecting at the center of the floor.

The American Tract Society Building, now in course of construction in New York, is to be about 240 ft. high, and the only provision to be made for wind bracing, besides the rigidity of connections, consists of lattice girders 4 ft. deep which surround the light well on one of the rear sides of the building at every story above the fourth. The building in plan is something like the letter U.

There is being constructed in Boston a building 10 or 11 stories high which, in his opinion, involves a novel construction. It is wholly a plate-girder construction; the corner columns are plate girders, and the intermediate ones are of riveted I section reinforced in the lower stories with plates and channels on thin flanges which in some cases have uneven areas to correspond to eccentric loading.

Each one of the four corners has a column, two of which are shown in cross-section (Fig. 1). The webs are from 35 to 51 ins. deep; there are six or eight angles which are reinforced by channels on the outside; these form continuous plate girders from the pavement to the top of the building. In the heavy wall girders in the long side of the

building, there is a further provision not common, in that allowance is made for the temperature expansion and contraction. The girders are web-connected to bracket pieces riveted on the sides of the columns, and have slotted bolt holes at one end; over the portal they are further strengthened by heavy bracket web plates. Mr. Skinner.

FIG. 1.

Fig. 2 is an end elevation of the building showing to scale in solid black the webs of the wall girders and plate-girder corner columns.

He called attention to the astonishingly poor constructions in New York which sometimes successfully withstand the winds and instanced a building in process of erection on Pine Street a year or two ago which had been built to the eleventh story. It was the poor modern cast-iron construction that is seen here too often, and the brick walls were built up to within two or three stories of the top tier of ironwork then set. After a severe wind, a plumb bob suspended near the top proved to be out of line nearly 2 ins. in a length of 80 ft. To correct this, diagonal transverse struts in vertical planes were put in at several floors and jack screws placed at the heels of the struts set up to right the building; this proving insufficient, wire rope diagonals were fixed in the same planes in the opposite direction, secured at both ends, and an attempt made to increase the tension by straining them by twisting, but this did not succeed. Eventually special connections were made and screw rod diagonals put in with sleeve-nut adjustment, and the framework was brought back apparently into place. Not a single column, however, was brought to a bearing; most of them had to be "shimmed up" $\frac{1}{8}$ to $\frac{7}{16}$ in. One column was adjusted by a small cut nail put in between the horizontal flanges of adjacent sections; when the weight imposed on the column was augmented by the wind pressure, it can be imagined where it would break off.

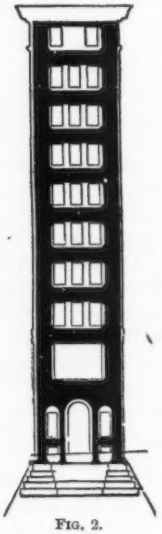

FIG. 2.

He alluded to some of the good constructions which must obtain in Chicago, to produce the result noted by Mr. Stebbins a year ago, when in a violent storm, some 60 or 70 miles an hour, a plumb bob suspended by a line extending over 14 stories of the Old Colony Building, then being erected, only described a vibration of $\frac{1}{2}$ in.; it seemed incredible that there should be so small a vibration, but the measurements were accurate, and the building must be astonishingly rigid.

CORRESPONDENCE.

Mr. Levering.

Mr. W. M. LEVERING remarked that he must disagree with the author in three of the conclusions stated in the paper, and briefly gave his reasons :

First.—The author said in "view of the uncertainty of the cast-iron columns and their imperfect connections for resisting transverse loads, it is but fair to consider them as hinged at each story in determining the stresses for knee braces, bending in columns, etc." And in referring to the strains in continuous steel columns, he stated that the strains on the knee braces would be reduced practically one-half; also, on the same page draws the conclusion that "buildings of this description should be provided with steel columns."

Taking the second example of a skeleton-constructed 17-story fireproof office building 32 ft. front, 100 ft. deep and 194 ft. high above the curb level, located on a corner, he considered the columns without reference to the material of which they are constructed as being continuous in one piece from foundation to top of building, and ascertained what the stresses would be at the different points of connection. It was needless to go into a calculation for the upper stories, since, as stated by the author, the maximum stresses on girders, bracing and connections, and bending in the columns, will occur in the floor system at the level of the adjoining roof and in tiers below it. Assuming that the wind pressure of 24 lbs. per square foot would have to be resisted by structural steel or ironwork and bracing, the horizontal force acting at the level of the fifth floor would be, as stated by the author, 55 500 lbs., which may be divided equally between the three columns in the cross-section of the building at that point.

He observed further, that the details of connection of side columns to girders with knee braces shown in the paper, the distance from the center of the girder to the bottom of knee brace being 2 ft. and the height of the story from center to center of girders being 11 ft., leave a clear, unsupported distance of 9 ft. between the bottom of the brace and the center of the girder of the floor below. If the columns are considered as rigidly connected to the girders, they may be taken as having fixed ends, with the point of contraflexure one-half way between the bottom of the brace and the center of the girder in the floor below, or $4\frac{1}{2}$ ft. from each. The total horizontal force of 18 500 lbs. acting on each column will have to be transmitted from the foot of the knee brace to the point of contraflexure, and from the point of contraflexure to the center of the girder in the floor below, and will produce a bending moment in the column of 18 500 lbs. multiplied by $4\frac{1}{2}$ ft. multiplied by 12 ins., equaling 999 000 in.-lbs. at the point of connection of the knee brace and at the center of the girder.

Mr. Levering.

Assuming the author's data, the dead load on the wall columns at the head of the fourth-story columns will be about 400 000 lbs., which, divided by the 54 sq. ins. in the cross-section of the column will give a uniform compression of 7 400 lbs. per square inch. The strain in the wall columns due to the action of the wind, considering the whole cross-section of the building as a girder standing on end, will be 55 500 lbs., multiplied by 72 ft., divided by 32 ft., which equals 124 000 lbs. direct compression on the leeward and direct tension on the windward column, which, divided by 54 sq. ins., equals 2 300 lbs. per square inch. In addition to this there was a bending moment of 999 000 in.-lbs. divided by 175 ins. resistance, which makes a compressive strain on the one side of the column and a tensile strain on the other side of 5 700 lbs. per square inch. This gives a total compression on the extreme fibers of the interior side of the leeward column at the knee braces of 15 400 lbs. per square inch, and on the exterior of 4 000 lbs. per square inch. In addition to this, assuming the live load (with the author) at an average of 50 lbs. per square foot of floor, a live load of 83 000 lbs. total will be produced, which, divided by 54 sq. ins., equals 1 600 lbs. per square inch, thus giving a maximum compression at the foot of the knee brace of 17 000 lbs. per square inch, and a minimum compression under dead load and wind only of 4 000 lbs. per square inch.

On the windward column there was, as before, a dead load compression of 7 400 lbs. per square inch, and a direct tensile strain of 2 300 lbs. per square inch (the column being considered as the flange of a girder of the full cross-section of the building); also a bending stress, as before, of 5 700 lbs. per square inch compression on the outside of the column at the knee braces, and of the same amount in tension on the inside of the column. This makes a total compression on the extreme outside fibers of 10 800 lbs. per square inch on the outside of the column, and a tensile strain of 600 lbs. per square inch on the inside. Adding the live load of 50 lbs. per square foot of floor, equal to 1 600 lbs. per square inch on the cross-section of the column, gives a maximum compression of 12 400 lbs. per square inch, and a minimum compression of 1 000 lbs. per square inch.

He then stated that the stresses in the foot of the column at the center of the girder of the floor below are the same as at the knee brace, but occur on the opposite sides of the column. The greatest tension in any portion of the exterior columns occurs when the wind and dead load only are acting, and amounts to 600 lbs. per square inch acting on 175 ins. resistance in the column, or, in other words, makes a total bending moment in the column at this point (which may be considered as the joint of the column) of 175 ins. multiplied by 600 lbs. or 105 000 in.-lbs. If the bolts connecting the head and base of the column are considered spaced 15 ins. apart, center to center, this

Mr. Levering.

remaining bending moment will produce a tensile strain on the bolts on one side of the column of 105 000 lbs., divided by 15 ins., equal to 7 000 lbs., which would be safely resisted by two $\frac{7}{8}$-in. bolts connecting the flanges.

Then, taking the middle column at the same floor level, the dead load as stated by the author is 250 000 lbs., and dividing this by 50 sq. ins. area of the column, there results a compression of 5 000 lbs. per square inch. Being in the neutral axis of the building when considered as a vertical girder, this column has no direct strain due to the wind, but has, as before, a bending moment of 999 000 lbs., which, divided by 147 ins. resistance of the column, produces compression on one side and tension on the other of 6 800 lbs. per square inch. The strains due to the wind and dead load are, therefore, a compression of 11 800 lbs. per square inch on one side of the column, and a tension of 1 800 lbs. per square inch on the opposite side. The live load of 165 000 lbs. on the column, divided by 50 sq. ins., gives a uniform compressive stress of 3 300 lbs. per square inch, thus making a total of 15 100 lbs. per square inch on one side of the columns, and 1 500 lbs. per square inch on the other, both in compression. The maximum tension of 1 800 lbs. per square inch, acting on 147 ins. resistance of the column, would make a total moment of 264 600 in.-lbs. tending to strain the bolts connecting the columns. If the bolts are placed 15 ins. apart as before, this will produce a strain on them of 17 600 lbs., which would be resisted by two $1\frac{1}{8}$-in. bolts on each side of the column.

Again, taking the stresses at the head of the second-story columns we find, by similar reasoning, a maximum stress in the leeward column of 19 200 lbs. per square inch, and a minimum stress of 6 000 lbs. per square inch, both in compression. On the windward column we have a maximum stress of 13 200 lbs. per square inch and a minimum stress equal to zero, and on the center column a maximum stress of 16 400 lbs. per square inch in compression, and a maximum tensile stress of 1 100 lbs. per square inch. The maximum tensile stresses are less in this case than at the fifth floor level.

He added that the total bending moment at the foot of the knee brace being 999 000 in.-lbs., a bending moment in the same direction on the foot of the column above of the same amount will make a total bending strain in the bracket of 1 998 000 in.-lbs., which, divided by 24 ins., will give the maximum horizontal stress to be transmitted by wind brace into the girder of 83 250 lbs. In addition to this, the wind brace has to transmit directly from the underside of the girder to the column the amount of direct horizontal stress at the level of the center of girder, amounting to 18 500 lbs., making a total of 101 750 lbs., which is the same as that found by the author for a discontinuous column, and not one-half of it, as stated. The total bending moment which

must be transmitted from the center column to the girder at its head has been assumed to be the same as that of the side column. If brackets are used at this point capable of taking both compression and tension, they will both act at the same time; consequently the strains in each will amount to half of that given for the side brackets. The total stress in each bracket at the center column being one-half of that in the bracket at the wall column, the point of contraflexure of the floor girder will be two-thirds of the span from the wall column, or $10\frac{2}{3}$ ft. Mr. Levering.

The shear in the girder at its point of contraflexure will be 101 750 lbs., multiplied by 2 ft., divided by $10\frac{2}{3}$ ft. or 19 075 lbs., which, multiplied by its leverage of $8\frac{2}{3}$ ft. and by 12 ins., will give 1 984 000 in.-lbs. bending moment at the end of bracket. This is resisted by a pair of 15-in. 180-lb. I beams, which have a resistance of 176 ins. and produces a stress of a little less than 11 300 lbs. per square inch on the extreme fibers of the girder.

The extreme maximum compressive strains in the fourth-story columns amounting to 17 000 lbs. per square inch, and in the second-story columns to 19 200 lbs. per square inch, are excessive in the case given, but it must be borne in mind that they occur only at the points where the columns are connected to the girders, and consequently no reduction for length need be made, and a small addition to their diameter would bring them within the prescribed limits of a factor of safety of six, if it was considered necessary to do so, without very greatly increasing their weight. As a wind exerting a force of 30 lbs. per square foot on any large surface is extremely rare, it is doubtful whether it would be necessary to calculate the column for this extreme case with a greater factor of safety than five. The strain of 11 300 lbs. per square inch on the cross-girder occurs only near the end, where there is little stress produced by a direct loading of the floors, and which diminishes rapidly as the center of the girder is approached, so that the girder being of uniform section, it is unnecessary to use, additional material to provide for this strain in the present case. The principal point in the whole matter is to make the detail connecting the cross-girder and column of sufficient strength to make the joint rigid.

He thought that with good design and workmanship, it was perfectly fair to consider cast-iron columns as continuous, since the dead load, even in the extreme case taken, very nearly overcomes the tensile stresses produced by the bending.

Second.—The stresses in the bracket and floor girders are independent of the continuity of the column.

Third.—It would be unwise to condemn cast-iron columns on the ground of their inability to resist transverse forces due to wind on large buildings, as they have decided advantages in price, in the shorter time required for their manufacture, and also in their ability to

Mr. Levering. resist rust, both on account of their material and the greater thickness of it used.

He added that in thus assuming the defence of cast-iron for columns of buildings he was well aware that he is taking the unpopular side of an argument from the point of view of the engineer, but, nevertheless, it was his opinion that for the special purpose for which its use was advocated it possesses special fitness which cannot be claimed for it in bridges and similar structures, for which engineers have, with good reason, condemned it.

Mr Breithaupt W. H. BREITHAUPT, M. Am. Soc. C. E., wrote that on data determined by observations covering many varied cases, and now generally acted on by engineers in bracing structures against wind, the assumption of a wind pressure of 30 lbs. per square foot of exposed surface is low; such pressure should reasonably be taken at 40 lbs. at least. On the other hand, it is reasonable to count on the resistance of the walls of the building, by carefully determining, as nearly as may be, from the dimensions, openings and material, what this resistance may be. To assume a uniform percentage for this resistance, which varies so much in different buildings, is, at best, a loose method.

The floors of a building are in nearly all cases stiff and continuous enough to more than provide for horizontal bracing; but the vertical partitions, as spoken of in the paper under discussion, are generally insufficient as bracing. As stiffening the structure throughout against vibrations from all causes, the method of vertical bracing by means of knee braces at all connections of beams to columns, in the manner of the old style of wooden framing, has much to recommend it. Such braces should not be under the beams only, but across all four of the angles in the plane of intersection of beam and column. It is generally easily put in, and does not obstruct space; the objection to it is that under unequal loading it induces bending moments in the columns.

Another method of vertical bracing is to have, in effect, vertical webs, not at every panel point, but at alternate panel points, or further apart, as the case may require, extending continuously throughout the height of the building, to the foundations. This bracing, either of rods or stiff, should preferably extend directly across the panels. When this causes detrimental obstruction, heavy gussets are required, and columns and beams must be reinforced for the ensuing bending moments.

A frequent weak point in iron-building construction is that the connections are inadequate. It is hardly necessary here to call attention to the waste of material this means in the way of nullifying the value of heavy sections.

In this connection it is pertinent to say something about the present tendency toward very high buildings. From the standpoint of possi-

bilities of construction there is nothing in the way of heights being still increased. In these structures the principal loads are practically static; the horizontal dimensions are generally such that stability, both of foundation and superstructure, can readily be secured, and the metal is generally so well covered by non-conducting material that temperature effects are not great. But from sanitary, economic and artistic points of view, the maximum appropriate height must be said in many cases to be already considerably passed. An isolated building with large open space on all sides may well be carried to extreme practicable height. But these conditions do not prevail with ordinary city buildings. Streets lined with such high buildings as are now being put up are converted, comparatively, into dark alleys, and the lower stories are, practically, dark, unventilated subcellars. This condition already prevails to a large extent, so that rents for the lower stories have had to be much reduced. A height of 120 to 140 ft., giving, say, 10-story buildings, should be the maximum. To this end there might now well be restrictive legislation. Mr Breithaupt

HENRY B. SEAMAN, M. Am. Soc. C. E., observed that the subject of wind bracing in high buildings has never yet received the consensus of opinion which it deserves. Mr. Seaman.

The practice in bridge construction is now well established at 30 lbs. per square foot of exposed surface with moving train, and at 50 lbs. per square foot for high trestles.

The practice of using 40 lbs. per square foot for buildings is perhaps as well authorized, but the general tendency in structures built without supervision is to lessen this provision and trust to endorsement by reference to similar structures "which still stand."

The reduction of wind pressure (from 50 lbs. per square foot for exposed trestles and high bridges, to 40 lbs. per square foot for buildings) is accounted for by the different character of the exposed surface.

Bridge work is made up of an accumulation of small exposed surfaces, which in calculations amount to but a portion of the area covered by the entire truss. In buildings, however, the entire area is calculated as exposed to the wind, while in fact the actual total pressure is not correspondingly increased.

The proposed practice of apportioning the resistance to wind pressure between the rigid partitions and the comparatively elastic iron or steel bracing, is of questionable wisdom. It is true that the partitions do offer resistance, and so does the iron bracing, but they do not act together. Several years ago a high building in Philadelphia, Pa., was being racked by the action of several engines placed in the upper story. In designing an iron framework to stop these vibrations, a complete system of bracing was put in, with horizontal struts at each floor and diagonal rods between. The system extended the full height of

Mr. Seaman. the building, and the rods were pulled to a strong initial tension. A better opportunity to test the efficiency of wrought iron, as compared with rigid partitions, could not have been desired. The results, however, showed clearly the inefficiency of wrought iron to act with the partitions, which, in spite of all precautions, continued to crack under the vibrations of the engines until the latter were removed.

This, he thought, demonstrates that we must either expect our partitions to take the entire strain, or design the iron work to take it in an emergency, should the partitions be destroyed.

He believed that the author's statement that sway bracing cannot be adopted is perhaps too sweeping. It cannot well be placed in the outside walls on account of the numerous windows, but it is needed there less than on the interior; and bracing can often be arranged in the partitions with no inconvenience to the design of the building. In this connection, it is interesting to note that the entire upper portion of the Chestnut Street half of the Drexel Building of Philadelphia, Pa., designed by Messrs. Wilson Bros. & Co., has not only a system of bracing, but a complete set of trusses, extending up through the partitions, and carrying the building over the Stock Board room and over the bank below. This plan of construction has been successfully adopted in several other buildings since that time.

He called attention to the question of the corrosion of wrought iron, and remarked that it is to be hoped that the design of iron structures will gradually undergo the change which has taken place in plumbing and other sanitary arrangements, where the entire system is exposed to inspection. If the details of the iron work are effectively arranged, exposed and decorated, they will present an attractive finish. Where columns must be embedded in masonry, cast iron is much less subject to corrosion than wrought iron or rolled steel.

He closed by referring to the importance of scientific design, both as to general arrangement and as to details of construction, particularly with regard to secondary strains.

Mr. Leverich. G. LEVERICH, M. Am. Soc. C. E. stated that the problem in engineering design comprised in the safe construction of the metallic framework of the high buildings lately erected, and conspicuous in this and other cities, when divested of extraneous considerations, is a comparatively simple one. The framework, consisting of lines of vertical columns, connected by and supporting horizontal beams placed in planes one above the other, the latter bearing certain uniform and other variable loads, and the whole to be subject at times as a united system to distributed horizontal forces, is in general a beam, or series of beams joined together laterally, with one end free and the other sustained. It is at once apparent, that such a structure without diagonal members, or their equivalents, which will transmit the stresses due to the resultant applied forces from their points of application

to the foundations, will be unstable; if, however, diagonals may be inserted, a positive solution is easy and need not here be further commented on. Mr. Leverich.

The question to be answered therefore is—what is a sufficient equivalent for the diagonals?

He then pointed out that if the spaces enclosed by the vertical and horizontal members of the framework be closely filled in with the materials comprising the outer and inner walls of the building, such, if sufficiently resistant, will serve as a means to transmit the diagonal stresses; this construction, however, is not permissible in many of the cases submitted to the designer. Generally the outer walls should be detached from the framework, and the inner walls must be placed and provided with openings, without regard to their possible work or efficiency as members of a composite structure.

The other equivalent for diagonals to prevent deformation of the framework is a system of braces connecting the horizontal and vertical members at their several joinings. A slight consideration of this device will show that thereby the beams and columns may be subjected to stresses transverse to their axes; in the former, perhaps, surely opposed by the dead loads they carry, but, in the latter, tending to cause flexure and in uncertain degree thereby to disable these long vertical members from sustaining the dead loads imposed.

Therefore, in conclusion of this treatment of the problem, it may be urged:

First.—That the columns be substantially continuous from foundation to top, and the beams in each direction securely joined to them.

Second.—That diagonals be inserted in as many of the several rectangular enclosed spaces as the case will permit.

Third.—That in all other spaces a brace be placed at each joining, dimensioned to enclose as large a triangle as practicable; also, that where this is done, the connected column be there proportioned to safely withstand the transverse as well as the axial stresses to which it thus may be subjected.

In practice, to solve the problem considered, the measure of the imposed forces must be known. The architect, builders and users of these buildings will fix with reasonable correctness the loads per unit of floor space to be sustained, and with these given, so soon as the design is outlined, the maximum vertical forces to be provided for may be computed with reasonable accuracy. Were these the only forces to be dealt with, a safe plan, consisting of simple vertical and horizontal members, could readily be made without the aid of an engineer. The measure of the horizontal forces, those due to wind pressure alone, and which make necessary the insertion of diagonal parts or their equivalents, is, however, yet indeterminate.

He called attention to the record referred to in the paper read

Mr. Leverich. on wind pressure at different dates at Central Park, in which such ranges from 49½ to 21¾ lbs. per square foot of vertical exposed area; and, taking these as authoritative, the author prudently says that "not less than 30 lbs. per square foot horizontal pressure on the exposed windward portion of high buildings should be provided for." Presumptively, the pressures quoted were deduced by some accepted formula from the velocities observed at the times mentioned. How fallacious such a procedure is may be learned by reference to a paper entitled, "An Examination Into the Methods of Determining Wind Pressures," * and a preceding paper, entitled, "Wind Pressure upon Bridges," † with discussions thereon. These monographs briefly record the knowledge on this subject up to the date of writing (May, 1881), and but little of real weight, based upon original and pertinent observation, has been added since.

The table of velocities and the consequent pressure of the wind, generally printed in text and table books, commonly known as "Smeaton's," from an early distinguished English civil engineer, are based on limited experiments, concluded 135 years ago, which he did not make, and upon conclusions therefrom, which he did not endorse.‡ It should be called Rouse's table. Other tables, based upon Hutton's and Hagen's deductions (subsequent to Rouse's time), have also been published. These three § are different, and each, for considerable velocities of the wind, is manifestly unreliable. Thus, according to them, one who faces a "strong gale" as he advances has to resist and overcome a force of from 18 to 12½ lbs. upon each square foot of his exposed vertical section. Ordinary window glass cannot withstand such a pressure, although it commonly does remain intact against such a wind.

He remarked that when the design for the Forth Bridge was in preparation, a series of observations, extending over a period of more than eight years, was made near the site of this structure with the purpose of determining the maximum wind pressures to which the bridge would be subjected.‖ Three anemometers or wind gauges of special construction were employed; one having an exposed surface of rectangular outline 20 ft. long and 15 ft. high, or 300 sq. ft. area, and another having an exposed surface of circular outline 1½ sq. ft. in area, were fixed to face always in one and the same direction and toward the west; a third, having also an exposed surface of circular outline of 1½ sq. ft. in area, was pivoted so as to always face the wind current. Each exposed surface was plane and closed; the test plate was hung in a frame, so as to yield under the wind pressure, the consequent movement

* *Transactions*, Am. Soc. C. E., Vol. X, p. 172.
† *Transactions*, Am. Soc. C. E., Vol. X, p. 139.
‡ *Transactions* of the Royal Society, 1759.
§ *Transactions*, Am. Soc. C. E., Vol. X, p. 176.
‖ See *Engineering*, February 28th, 1890, p. 221.

being opposed by springs; the registration was effected by a system of chains, rods and levers, actuated by the moving plate and terminating with an index hand, which oscillated before a graduated scale. The arrangement was crude, and its indications not precise. The observed results from the operation of these three anemometers appear in the following table—the same being a record of maximum pressures per square foot, except the last two columns, which give comparative values of the observations, reduced to a plane normal to the direction of the wind, that for the large gauge being unity. Mr. Leverich

Dates.	Direction of the wind.	Observed. Maximum pressures. Pounds per square foot.			Computed—Normal to the Wind.		Comparative values—pressure on large gauge being 1.	
		Large fixed gauge.	Small fixed gauge.	Small revolving gauge.	Large fixed gauge.	Small fixed gauge.	Small fixed gauge.	Small revolving gauge.
		a.	*b.*	*c.*	*d.*	*e.*	*f.*	*g.*
Oct. 27th, 1884.....	S. W.	18	23	29	25.5	32.5	1.28	1.14
Oct. 28th, 1884.....	"	19	29	26	26.9	41	1.53	1.07
March 20th, 1885...	W.	17	25	30	17	25	1.47	1.77
Dec. 4th, 1885.....	"	19	27	25	19	27	1.42	1.32
March 31st, 1886...	S. W.	19	31	26	26.9	43.8	1.63	0.97
Feb. 4th, 1887.....	"	15	41	26	21.2	58	2.73	1.22
Jan. 5th, 1888.....	S. E.	7	16	27	9.9	22.6	2.29	2.73
Nov. 17th, 1888.....	W.	27	41	35	27	41	1.52	1.3
Nov. 2d, 1889.....	S. W.	12	34	27	17	48.1	2.83	1.59
Jan. 19th, 1890.....	"	16	28	27	22.6	39.6	1.76	1.19
Jan. 21st, 1890.....	W.	15	38	26	15	38	2.53	1.73
Jan. 25th, 1890.....	S. W. by W.	18	24	27	47	62.7	1.33	0.57

It is to be understood that in this table, columns *a*, *b* and *c* record the pressures as observed; columns *d* and *e*, the mathematical resultants of the fixed gauge pressures upon a plane normal to the wind; and columns *f* and *g*, the comparative values of the pressures recorded in columns *c*, *d* and *e*—those in *d* relating to the large fixed gauge being unity. It will be noticed that column *f* also exhibits the relation between the pressures observed on the fixed gauges, recorded in columns *a* and *b*.

He stated that but little of practical value is to be deduced from this table; it records that the pressures on the small fixed gauge are from 1.28 to 2.83 times those on the large fixed gauge; that the greater pressure on the latter is 27 lbs., from a wind blowing fair on its face, and that the assumption under which the normal pressures written in columns *d* and *e* were computed from the observed pressures

Mr. Leverich. due to wind blowing at an angle to the test planes is invalidated by the discrepant and unreasonable results thus obtained.

In these latitudes, almost invariably when high winds have caused destruction, it has been observed that the glass in the windows, the lighter, vertical walls, and sometimes the roofs, have been first thrust outward, indicating that the moving forces were consequent upon a difference at the time in pressure between the air within and the air without the structure, the latter being much the smaller. This seems to show that the current assumption as to the action of high winds, that they produce a great inward pressure on structures, is erroneous. Engineers and constructors should know the amount of pressure per unit of exposed area of structures produced by high winds as definitely as they know the limits of endurance of the materials they employ in these structures. Doubtless if this had been earlier recognized, before now, conclusive observations would have been made, and exhaustive deductions therefrom, fitting the results for ready application in common practice, would have been published.

Generally the data governing this matter, accepted as authoritative, have been based upon records of velocities of the wind at times of notable disturbance, measured in a manner which did not make quick registry of the effects of extreme waves or pulsations of movement frequently occurring. Two sources of error are consequent; one, the uncertainty of the relation between the observed velocities and the resulting pressures of free currents of an elastic medium; and the other, the difficulty of eliminating from the recording mechanism the disturbing tendency of the inertia of its several parts, acting as it does, to indicate greater or less effects than had been produced at the precise times of observation. The first, as already stated, is one that discredits the several methods of deducing pressures from velocities; the second may be removed by the use of a suitable anemometer, which will measure and record the pressure itself in all its varient phases, and as exerted under the several conditions of effect on the outer surfaces of existing structures. This would include exposure to the wind of plane surfaces, closed and latticed, angular and curved, normal and inclined to the axis of pressure, each of comparatively large area; also observations of the effect upon each surface when it is shielded, and the resulting action of the wind thereon is caused by the partial vacuum developed within the consequent vortex or eddy on the leeward side.

He added that years ago, in the prosecution of an important and conspicuous public work, a large part of which is exposed to all the fitful winds prevailing in this northern latitude, what provision in its design should be made that the structure to be erected might without doubt safely withstand all assaults of the wind was a perplexing as well as an important problem. To aid in solving it, an

instrument was planned to measure and record the pressures of the wind upon exposed surfaces of large area, diverse forms, and generally like those embodied in engineering construction; each under varying conditions, and either normal or inclined to the axis of pressure. Before, however, its details were fully worked out, the work mentioned had progressed well toward completion, and the nascent scheme was consigned to limbo. Mr. Leverich.

From an entry in a note book, under date of January 13th, 1882, he had compiled a brief general description of the anemometer devised. In its design an attempt is made to suspend the plate, having an exposed surface to the wind, so that it may without friction in yielding move but a small space, and that such movement shall induce a pressure upon a column of light fluid, the amount of which shall be measured either by an opposing column of heavy fluid or by other simple mechanism. To a frame of small iron pipes, at one end and normal to its axis is suspended one of the several shapes of test plates, each of thin sheet metal, the surface of which is to be exposed; and at the other end is attached a vane, also of thin sheet metal, of dimensions sufficient to keep the exposed surface always square against the wind, as the whole rotates about a common center of gravity, on a universal pivot, in any plane within 30° of the horizontal. This pivot is on a vertical hollow shaft which revolves with the frame. The test plate is suspended to the frame at four points by strips of sheet metal thin enough to flex somewhat without perceptible resistance, and broad enough to have the requisite strength. Near each of these points, attached to the leeward side of the test plate, is a short piston closely fitted to and slightly entering a short cylinder attached to the frame; each with a thin metal diaphragm connecting the two, and with slight relative movement of these parts acting as a frictionless packing. A small branch pipe from each of these cylinders leads to the vertical shaft, which serves as a main pipe.

The apparatus thus far described is to be erected on a tripod of iron pipes which projects from the roof of the observatory building at a height above erratic or cross currents of air, and from it the vertical shaft is continued down into the building and to a convenient place for observation. To reduce the pressures, and to correspondingly increase the movement of the heavy registering fluid, a transforming device is used. This consists of a small piston fitted into the lower and free end of the hollow shaft or transmitting tube as a cylinder, and a second and much larger piston, fitted to and entering slightly into another and short cylinder closed at the other end; these two pistons and their cylinders having a common axis, and each pair made fluid tight by a thin metal diaphragm, as before mentioned. To the second cylinder is connected a vertical tube of small diameter, reaching upward and open at its free end.

Mr. Leverich. He omitted a description of sundry details required to provide for an adjustment of the apparatus necessary to insure its correct and uniform operation, also to reduce the friction of its moving parts. Previous to the use of the anemometer, and when there is no wind, the branch and main tubes are to be completely filled with a light transmitting fluid non-congealing at the place of observation, as alcohol, glycerine or the like, and then closed. The larger reducing cylinder and the registering tube is to be filled with a heavy registering fluid, as mercury, up to zero point, on a scale attached to the latter.

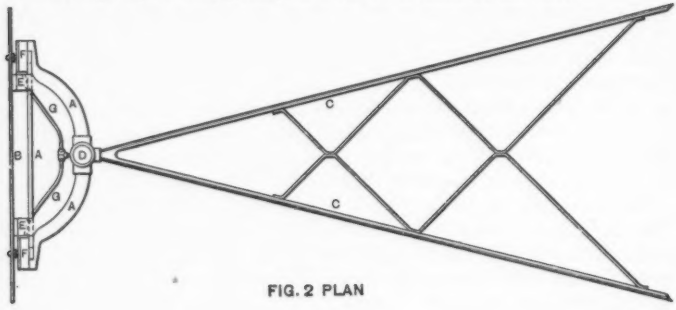

FIG. 2 PLAN

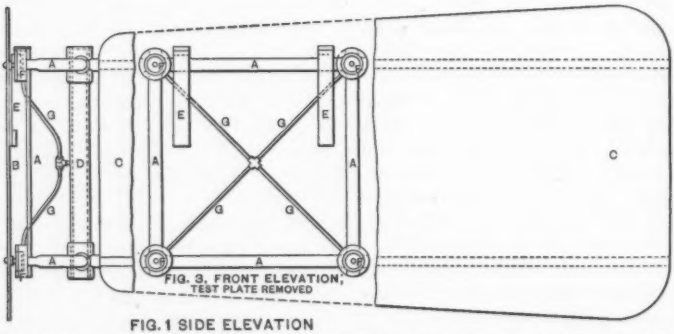

FIG. 3. FRONT ELEVATION, TEST PLATE REMOVED

FIG. 1 SIDE ELEVATION

A simple arrangement of the several parts, designed, however, to revolve in but a single horizontal plane, is shown by the illustrations; Fig. 1 being a side elevation; Fig. 2, a plan, and Fig. 3, a front elevation, the test plate having been removed. In these, *A* is the frame; *B* is the test plate; *C*, the vane; *D*, the hollow revolving shaft and main pipe; *E*, the suspension plates (of which but two are drawn); *F*, the transmitting pistons and cylinders, and *G*, the small branch pipes.

The operation of the apparatus, which he had thus in general described, is as follows: Pressure from a wind current on the exposed test

surface, causing a slight movement of the latter, induces pressure on and a corresponding movement of the transmitting fluid. This pressure at the transformer is opposed by the registering fluid, the movement of which is multiplied, so that its range of vertical oscillations is sufficient to permit close measurements on the scale attached. Instead, however, of this simple method of observation, requiring as it would the constant presence of an attendant, the anemometer was so designed that it should be self-recording, itself registering continuously the compass points and deflections from the horizontal, as well as the varying pressures of the wind currents as they sprung up and fell away. Mechanism comparatively simple, to make the several records, was worked out in the design as first made. Mr Leverich.

It was proposed that the test plates should have a transverse plane section of 10 sq. ft.; that their pistons should be $2\frac{1}{2}$ ins.; the two transformer pistons $1\frac{1}{16}$ and $3\frac{1}{2}$ ins., and the registering tube, $\frac{1}{4}$ in. each in diameter. Then, under a maximum pressure of the wind on the test plate of 100 lbs. per square foot, it would yield and flex its suspending plates but $\frac{1}{20}$ in., and the mercury would rise in the registry tube 10 ins., or $\frac{1}{10}$ in. per pound of wind pressure indicated.

Guy B. Waite, Assoc. M. Am. Soc. C. E., in closing the discussion, stated that experiences gained in the examination of building construction as submitted to the Department of Buildings of New York City, had led him to present his paper. Mr. Waite.

He had observed that, with few exceptions, the most important new buildings were not receiving systematic consideration for lateral stiffness and stability, and that it was often necessary to criticise such construction for defects of this kind, and to substantiate the objections by actual demonstration.

He had chosen for discussion examples which, in general dimensions, design, sizes of material and manner of location, were similar to plans of proposed buildings which had come under his observation, the only changes made being the insertion of brick mullions in place of a more nearly open front for the first example discussed, and in the substitution of the knee brace and column and girder connections shown in the 17-story building, instead of inferior ones found on the original plans.

As low a wind pressure as could be sanctioned at the present time by any standard authority was taken in discussing these examples, and credit given to every feature of construction which could reasonably be claimed as aiding the protection of the buildings from wind and in enabling them to resist lateral forces. It was considered that by so doing unprofitable issues would be avoided and needed discussions of real conditions would be induced. In the first example (non-fireproof building, 23 ft. front and 85 ft. high), he had desired to call attention to the necessity of some system of stiffening for the front and

Mr. Waite. the rear of such buildings where sufficient rigidity cannot be found in other parts of the construction. The second example (a 17-story building), having knee braces for lateral stiffness, was especially taken for the encouragement of a systematic consideration for the designing of this now common system of bracing high building structures.

He believed that whenever a simple and more direct method than knee bracing can be adopted, it is certainly preferable. That diagonal sway bracing has been used, and can still often be used, is well known. The impression which he had wished to convey when stating verbally that sway bracing could not be used in building construction was simply to call attention to the difficulty of adopting it to the majority of designs. As the part of the engineer in the designing of buildings is to deal with the conditions submitted to him by the architect, he must often be contented with means which otherwise he would not favor.

The manner of depending upon all of the elements composing the structure, such as walls, partitions and the special connections and bracing to act in conjunction in resisting lateral forces, he did not believe to be practicable unless great care is exercised in the construction with that purpose in view.

Their values had, however, been carefully determined for the discussion of this paper, in order to satisfy those who might otherwise use it as a text for dissension from any discussion which could be advanced on wind bracing without considering all of such elements. The calculations for the resistance of walls and partitions were systematically made by the principles of moments; the resisting moment of the dead weight of walls being counterbalanced by the overturning moments of the tabulated wind pressures given in the original paper. The amount of resistance of the brick mullions was determined by assuming the imposed stresses to act in a diagonal direction, and the resistance of the partitions was ascertained by calculating their resisting moments, the resistance acting in a vertical direction. Results of calculations only were given in the original paper.

He remarked that the very interesting discussion of Mr. Levering was instructive in showing the manner of determining the stresses in columns, girders, knee bracing, etc., but that although his determinations of the stresses imposed on continuous columns rigidly fixed at each connection and at floor levels are (with but few exceptions) correct, they seem foreign to any proof that cast-iron columns, as now used in high building structures, should be considered as continuous.

He called attention to the fact that Mr. Levering assumes, for a premise, that the columns are perfectly continuous and rigidly fixed at each floor level, and on that assumption determines certain stresses, from which the conclusion is reached that they may be taken as continuous. The transverse stresses on the columns are about twice as great when taking them simply supported at points of sup-

port as they are when considering them as fixed at these points, and therefore the stresses found will be vastly changed from Mr. Levering's determinations when considering the columns as hinged, as assumed in the original paper. Mr. Waite.

Mr. Levering finds stresses on columns as follows :

	Pounds per square inch.
Windward columns, maximum compression.....	12 400
" " tension.....	600
Center columns, maximum compression.....	15 100
" " tension.....	1 800

Now, if these columns had been considered as hinged, the corresponding stresses would have been :

	Pounds per square inch.
Windward columns, maximum compression..	12 400 + 5 700 = 18 100
" " tension	600 + 5 700 = 6 300
Center columns, maximum compression.....	15 100 + 6 800 = 21 900
" " tension.....	1 800 + 6 800 = 8 600

It had not been his intention to condemn the use of cast-iron columns in building construction, but simply to assist in the development, from actual data, of the amount of stresses imposed on them under certain conditions, and to show that whenever it is not reasonably certain that the columns will be able to stand under those conditions, it is then but wise to recommend such materials as are best adapted to the case.

He further stated, that in developing for the stresses obtained in the original paper, the assumptions made must be conceded to be all favorable to the construction, thus :

First.—Only the actual exposed area above the adjoining building was taken as acted on by winds.

Second.—It was taken for granted that the adjacent building was capable of resisting the thrust of wind pressure (acting on the structure under consideration) when blowing towards the adjoining building.

Third.—It was assumed that all parts of the walls and columns were perfectly plumb, and that there were no inherent lateral stresses in the members of the structure under consideration.

Fourth.—All parts of the entire structure (dead loads, partitions, connections of columns and girders, and special bracing) were assumed to act in perfect unison in resisting lateral forces—which would be a remarkable circumstance, indeed.

Fifth.—The maximum distributed wind pressure for the highest buildings was taken at 30 lbs. per square foot, which is the lowest given by any known standard authority. The formula used for the

Mr. Waite. reduction of wind pressure from velocity was recently derived from experiments made at Mount Washington, and may be found in Johnson's "Theory of Framed Structures," as follows:

$$P = .004\ v^2 \qquad v = \text{velocity in miles per hour.}$$

This formula gives less pressures for corresponding velocities than most of the older formulas.

It seemed to him that unless all of the circumstances assumed should be favorable the transverse column stresses would be materially increased. In an ideal case it may be assumed that cast-iron columns will act with continuity, but as they are actually constructed such an assumption would be unwarranted, and, in the present case, so much on the side of danger as to be hazardous, for the following reasons:

First.—In the plumbing up of cast-iron columns the main bearing is often brought to one side of the center, giving an eccentric bearing, and when "shims" are introduced to keep the column plumb, they not infrequently either make eccentric bearings or they give a bearing on the neutral axis—which is equivalent to a hinge bearing.

Second.—The casting of the columns is often very uneven; it may have a thin portion at the very point when the maximum stresses are imposed.

Third.—Cast iron is subject to internal strains, which, if sufficiently assisted by imposed stresses (especially those acting transversely) may cause rupture.

Fourth.—Cast iron is subject to defective homogeneity, has very little, if any, elasticity, and almost an absence of ductility. In assuming such columns as non-continuous, it will be acting in the direction of safety.

It has recently been learned that the government of Prussia requires that the safe-carrying capacities of cast-iron columns shall be determined by considering them as having hinged bearings.

He further stated that Mr. Levering's determinations would tend to show that there is no advantage gained by the use of a solid knee brace instead of a simple angle brace, that the stress in the brace and girder is the same whether a solid brace rigidly fixed to the column and girder throughout is used or simple angles fastened to a particular part of the column and girder only. While his calculations for the determination of stresses in columns, girders and brace may not be in serious error, practically considered, his assumption, that the point of contraflexure in the column is at the same place when it is left free to deflect between the foot of the knee brace and the girder connection, as it would be if that part of the column were rigidly fixed in a vertical plane by a solid knee brace, is in violation of the laws governing the theory of flexure.

With the floor girder rigidly fixed (as assumed by Mr. Levering) there will be three points of contraflexure, one near each end of the

girder and one between the points of attachment of the braces (which he takes for discussion); the latter point of contraflexure will not be accurately found by the simple laws of proportion assumed. In determining the stresses in the floor girder Mr. Levering assumes that the entire stress is carried through the stiffening angles of the brace, which is not the case in the solid knee brace assumed in the original paper as more fully discussed hereafter. Mr. Waite.

It is true that the aggregate stresses imposed on the brace and the connections as a whole are about the same, whether it be a solid brace or a simple angle brace. The disposition and distribution of the stresses will, however, be entirely different in the two cases. The solid knee brace rigidly riveted to the steel column (considered in the original paper with the solid knee brace) and to the floor girder taken together with the riveted connection of the floor girder to the steel column, will make the entire adjacent material thus jointed together act as a single piece in taking up stresses imposed on any of its parts. Although the exact manner of the distribution of imposed stresses to individual members cannot be clearly analyzed it may be taken for granted that in all of such braces (some of which are subjected to tension and some to compression) the stress imposed at the level of the floor girder, by the bending in the column below the point of contraflexure, will be taken up and cared for by the column and girder connection and that part of the knee brace adjacent thereto, while the stress imposed by the bending of that part of the column above the point of contraflexure will be mainly carried through the stiffener angles forming the lower part of the solid knee brace. Therefore these angle stiffeners need only be made about one-half as heavy as when they compose the entire brace. No one wishes to go to the extra expense of making solid knee braces unless they have decided advantages over the simple angle iron brace ; and he felt warranted in concluding that not only do they present a more workmanlike class of construction than the simple brace, but the extra material added is not lost, as it takes an active part in performing the functions required of the brace.

He thought that steel skeleton-constructed buildings had not yet passed through the experimental stage, and all doubts as to the efficiency and durability of all parts used in their construction had not been dissipated, and also that the preservation of steel and iron under conditions sometimes favoring corrosion is a subject which should receive immediate and serious consideration, but about which very little is positively known at present.

He believed, with Mr. George A. Just, that much useful data may be derived from a careful study of old existing buildings, and that encouragement should be given to experiments for deriving a positive preventive against deterioration by corrosion of iron and steel in buildings.

Mr. Waite. The query of Mr. George B. Post concerning the effects of unequal settlement in a skeleton-constructed building he thought pertinent and deserving of serious thought. If the foundations of such buildings are carefully planned and constructed there need be little fear of detrimental settlements. If, however, such settlement does occur with a rigid steel frame, the derangement could only be effectively remedied by raising the settled portion and blocking it up in its original condition. The simple righting of a steel skeleton building when the joints are not rigidly fixed is a comparatively simple matter, and can be done by the insertion of diagonals from floor level to floor level and drawing the frame into a true vertical position.

Although most present buildings have withstood prevailing winds, he did not think it followed that the new type of high construction would not be subjected to conditions requiring entirely new and advanced considerations for making them permanently safe; sufficient time to test practically the efficiency of this character of construction has not elapsed. He called attention to the statement often made in the endeavor to show practically that the ordinary assumptions for wind pressure are excessive, that window glass would be ruptured by much less pressures than taken for the buildings, and stated that by actual computation on a $\frac{3}{16}$-in. glass, 36 ins. wide, fixed at its two ends (as well-puttied window glass is held), it is able to sustain pressures in excess of any ever assumed as acting against a building structure.

In conclusion he thought that the remarks of Mr. Just and Mr. Leverich point in the right direction for the determination of such reliable data as can be made of universal assistance in definitely formulating a treatment for the perplexing question of wind bracing for high buildings.

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TRANSACTIONS.

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746.

Vol. XXXIII.—March, 1895.

THE RELATIVE EFFECTS OF FROST AND THE SULPHATE OF SODA EFFLORESCENCE TESTS ON BUILDING STONES.

By LEA MCI. LUQUER, PH.D.

READ FEBRUARY 20TH, 1895.

WITH DISCUSSION.

A series of careful investigations to develop the relation between these two methods of testing building stones cannot fail to be of interest. Both methods have been used for many years for estimating the probable resistance of stone to weathering. The effects of frost have been obtained in various ways, but it is only recently that the establishment of cold storage rooms, where there is plenty of space and where almost any desired temperature can be maintained, has made it possible to carry on these tests conveniently and accurately.

The experiments have been generally only approximate, and the results recorded in a vague way, depending on the appearance of the stone after treatment and the presence of cracks, flaws, or detached particles in the vessel.

R. G. Hatfield* made experiments on the effect of frost on sandstones by exposing specimens in covered vessels, filled with water, from December to April. These specimens were alternately frozen and thawed many times.

Professor J. Henry† states that freezing tests were made 50 times on marble selected for the White House. He also says that tests made by the use of freezing mixtures are very laborious and must be repeated many times before results show.

Braun‡ gives the following method of getting at the probable effect of frost. He "institutes a comparison between the strength of extension of the material and the force of the solidifying water, assuming that a material is not capable of resisting frost when the former is less than the latter."

Bauschinger‡ used a more practical process, "consisting in the exposure of the material 25 times to frost in the open air, the strength before and after the test serving as a guide to the resisting power."

Blümcke,‡ in reviewing the different processes for getting at the probable effect of frost, says they all subject the stone to unnatural conditions and are more or less complicated.

The following method was used by him: Specimens were brought to a very low temperature, by being surrounded by a freezing mixture, were then placed in a vessel, and covered with water and left for three hours. "When taken out the stones were covered with a coating of hoar frost, and if then left for some time in water, a loosening of the small particles was perceptible in the portions not capable of resisting frost." The process was repeated until marked disintegration was noted.

Some tests on sandstone by this method are recorded.

Prof. Wilber‖ tried the effect of subjecting specimens to freezing and then to a temperature of 212° Fahr. in an air bath. The specimens were saturated with water in every case before being put in the vessel surrounded by the freezing mixture. Seven repetitions of this test were made on each sample. His results were as follows: Granites, marbles and limestones were unchanged, sandstones in some cases were slightly affected.

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*Am. Jour. Sci. (II), Vol. XXII, p. 30, 1856.

†Trans. Am. Soc. C. E., Vol. II, p. 148, 1873.

‡Van Nostrand's Eng. Mag., Vol. XXXIV, p. 44, 1886.

‖Tests by Prof. F. A. Wilber, Bull. N. Y. State Museum, Vol. II, No. 10, Sept., 1890, p. 358.

G. P. Merrill* states that the best way to determine the probable effect of frost is to expose specimens of stone, saturated with water, to the actual action of frost, and note the disintegration.

As in many cases, it is almost impossible, or very inconvenient, to get the real action of frost, other methods have been used for producing the desired effect.

The expansion of easily crystallizable salts has been used as a substitute for freezing water, and seems to have been considered at first as about equivalent in action, the salts, in passing from the liquid to the solid state (crystallizing), producing the same rending, shattering or disintegrating effect as water in freezing.

Brard's process, using sulphate of soda (Glauber salt), is the most common of these methods.

M. Vicat,† Ingénieur des Ponts et Chaussées, in 1821 made some experiments with sulphate of soda on mortars and stones. He found that stone which had been exposed to weathering and had resisted the action of frost very well, was noticeably affected by sulphate of soda (saturated hot) after 48 hours. Hence, he claimed that good stones might be rejected if tested in this way. He called attention to the fact that the action was considerably less when, instead of boiling the specimens for half an hour, they were boiled for only 10 minutes.

Prof. J. Henry‡ stated in 1856 that Brard's method was much used, the theory being that a cold saturated solution of sulphate of soda acted on the rock like freezing water, but that recent investigations of Dr. Owen lead us to doubt that its effects are exactly the same as natural freezing, the effect appearing to be chemical as well as mechanical. The experiments were made on porous stone and checked by actual freezing.

C. P. Townsley‖, under the head of external agencies producing weathering of stone, such as frost, says that the artificial test recommended by Brard is the most successful of all other methods. He recommends a cold saturated solution of sodic sulphate, and boiling the specimens for 30 minutes, to get complete saturation. Then suspend specimens and allow them to effloresce in a cool, damp room.

* Building and Ornamental Stones, p. 334, Washington, 1889.
† Annales de Chimie et de Physique, Vol. XXXVIII, p. 170.
‡ Testing Building Material, Am. Jour. Sci. (11), Vol. XXII, p. 30, 1856.
‖ Van Nostrand's Eng. Mag., Vol. XIV, p. 537, 1876.

Soak specimens in solution and let them effloresce again, continuing the tests for a week or more, and the amount of earthy material found in the vessel below will show the action of the sulphate or the corresponding action of the frost. This evidently assumes similarity of action of artificial efflorescence and frost.

The average results on the specimens tested seem to show that marbles and limestones withstood the action of sulphate of soda a little better than granites, and still more so than sandstones.

R. G. Hatfield* states, in relation to the sulphate of soda or artificial freezing, that "the specimens were immersed in a saturated solution for one day and night and the apparent action noted in a scale of from 1 to 12, the larger numbers indicating the greater action of the soda. Other specimens of the same stones were surrounded with small pieces of ice and left for the same length of time. These, when examined, seemed to be affected nearly to the same degree as those submitted to the sulphate of soda test."

C. G. Page† has conducted in this country the most important series of experiments with Brard's process in reference to the selection of material for the Smithsonian Institute Building of Washington. Observations recorded show that sandstone was in most cases much more affected than marble, and that the few specimens of granite tested were acted on in about the same way as the marble. The coarse Pleasantville marble seems to have been more affected than the close-grained varieties. The experiments lasted for four weeks, the specimens being boiled for half an hour and then hung up to effloresce, this process being repeated every day.

Professor J. C. Draper's‡ experiments with sulphate of soda proved that Nova Scotia stone was affected much more than brownstone and very much more than brick. "These results," he states, "also tend to show that frost is not the main agent of the initial disintegration in the climate of New York, since it is not the Nova Scotia stone, but the brown stone which suffers the most severely and rapidly from decay." This inference seems to indicate that Professor Draper regarded the artificial sulphate of soda efflorescence as equivalent to to the action of frost. The tests were carried on for eight days, and the disintegration recorded by loss in weight in parts in 10 000.

* Trans. Am. Soc. C. E., Vol. II, p. 143, 1873.
† Hints on Public Arch., by R. D. Owen, p. 119.
‡ Tenth U. S. Census, Art. on Building Stones, by Hawes & Merrill, p. 384, 1880.

G. P. Merrill* mentions that Brard's process, modified by d'Héricart and de Thury, is thought to act "in a manner somewhat similar to that of water when freezing."† He also states, however, that "this process is not now in general use, as experiment has shown that the salt exercises a chemical as well as mechanical action, and produces results somewhat at variance with that of freezing water."

The objection to Brard's method, or any other artificial method, is that stone, in nature, is not apt to be subjected first to the temperature of boiling water and then to freezing.

In view of the evident difference between the effects of these two methods of testing building stones, this paper has been undertaken to investigate the relation of one method to the other, when carried on under the same conditions and extending over the same periods of time. Theories may be advanced as to whether in the sulphate of soda test there is chemical combined with mechanical action, simply stronger mechanical action, or almost entirely a chemical action. But an investigation into the chemical actions possibly going on is outside the province of this paper, and the resulting facts alone will be given.

The experiments were made on carefully selected building stones. Great care was taken in preparing the specimens, and, unless otherwise stated, they were selected from fresh undecomposed stone. The samples subjected to the two methods of testing were in all cases adjoining pieces of the same stone. The specimens were not sawn out in cubes, as the crushing strength was not afterwards to be determined, but at the same time the rending and shattering effect of hammering was avoided as far as possible. The specimens were prepared with the "rock cutter," used for trimming minerals, and were made approximately of the same size and rectangular in shape.

They were then carefully examined and all sharp edges or loosened particles removed. Finally, they were brushed over with a stiff brush, dried in an air bath at about 120° Cent., and carefully weighed. In order to get the loss very exactly small specimens were used, and the weights taken with great accuracy on a chemical balance. The loss was obtained by the difference in weight before and after the experiments. The usual way in these investigations has been to deal with larger specimens, and get at the loss by weighing the detached particles

* Building and Ornamental Stones, p. 334. Washington, 1889.
† Chateau. Tech. du Bâtiment, Vol. I, p. 262.

very carefully, the exact weights of the larger specimens not being of so much importance in calculating the percentage of loss.

The experiments were commenced with the idea of getting the losses by the latter method; but many difficulties were encountered in obtaining accurate results, and the method was abandoned in favor of getting the losses by reweighing the specimens after treatment and subtracting the latter weights from the former. The 14 duplicate specimens selected for trial covered the different kinds of stones commonly used for building, such as marble, limestone, granite, gneiss, trap, sandstone and brick.

Outline of Method Followed in Making the Sulphate of Soda Efflorescence Test.

The process employed was Brard's method, as modified by d'Héricart and de Thury. *

A saturated cold solution of pure sulphate of soda (Glauber salt) was used, great stress being laid by de Thury on the solution being saturated cold. Experiments made by the French Inspector-General of Quarries have demonstrated the fact that stones which will resist frost and the action of a cold saturated solution of lye are entirely broken up by the action of a hot saturated solution. Experiments made by Dr. Vulté, of the School of Mines, on concentration of soda solutions, developed the following fact: When getting a hot saturated solution of this salt the temperature of the solution will rise above 212° Fahr. (100° Cent.), and will keep going up, until finally it passes what may be called the "critical temperature," when decomposition will take place, SO_2 being driven off and Na_2O being left behind, which may have the solvent action of an alkali on the silicates in the rocks. Decomposition will apparently also take place in the case of the limestones. The normal cold saturated solution of sulphate of soda is of course neutral. Former investigations all demonstrate the necessity of using a cold saturated solution.

The specimens, which had been carefully prepared, brushed, dried and weighed, were boiled in the sulphate of soda for half an hour, in order to get complete saturation. At the end of the half hour, it was noticed in every case that the solution was slightly alkaline, although at the start it had been neutral. In order to prevent any continued chemical action, the beakers were emptied, the specimens rapidly

* Chateau. Tech. du Bâtiment, Vol. I, p. 272.

washed with water, and the beakers immediately refilled with the neutral sulphate solution. After soaking for several hours, the specimens were hung up by threads, and left for 12 hours (during the night) in a dark room.

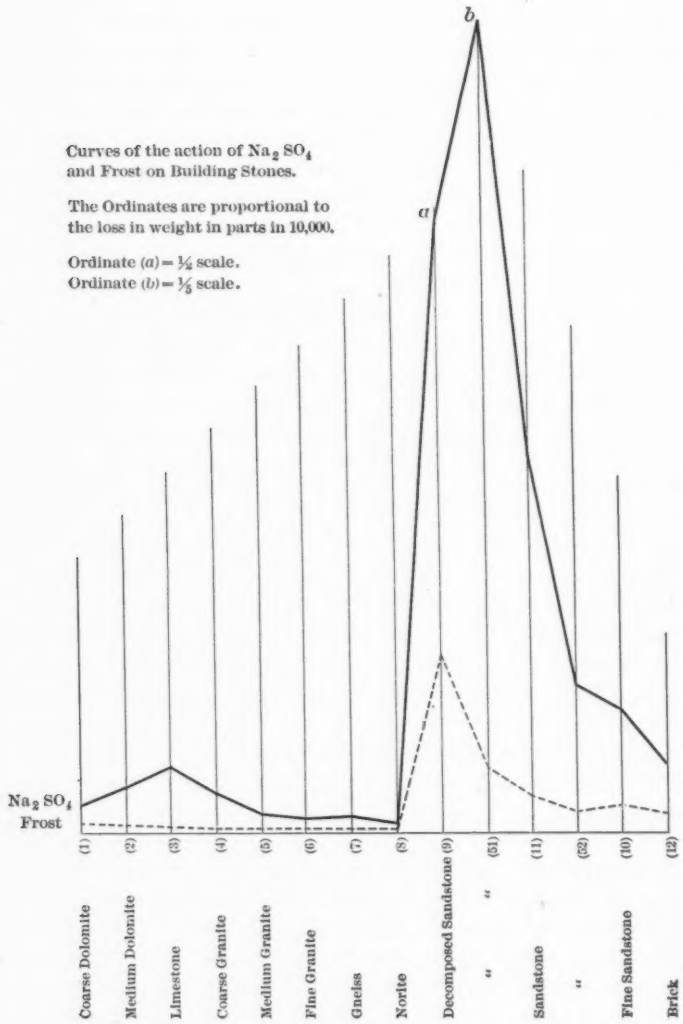
In the morning all the specimens were covered with an efflorescence of the white sulphate of soda crystals; they were then allowed to soak in the solution during the day and again hung up at night. Effloresc-ing for about 12 hours and soaking for about the same time constituted a period. The experiments lasted for eight periods, and were conducted in this way in order to make them correspond with those made with freezing water, as in the cold storage room the specimens could only be changed night and morning.

In two cases the specimens were allowed to effloresce for 36 instead of 12 hours, to insure thorough action of the salt. The experiments thus really lasted for 10 days. It was deemed that eight periods or days were sufficient, as de Thury states that if a specimen is acted on by this method of testing, the effect will be noticed in five days. The general opinion of others seems to be, also, that a week or eight days is long enough to obtain good results. During the tests the solution was renewed from time to time, and appeared to remain neutral. The tem-perature of the room varied from 60° to 70° Fahr. (18° to 21° Cent.). Those specimens most affected began to show the disintegrating action of the solution very early in the course of the experiments. At the end of the 10 days the specimens were sprayed with the stream from a wash bottle to remove any adhering particles, washed in water to remove the sulphate of soda, carefully dried in an air bath at about 120° Cent., and weighed again.

The difference between the weights was taken as the loss due to the action of the sulphate of soda. The results are given in tabular form.

Outline of Method Used in Making Freezing Tests.

These tests were made in the same manner as the sulphate of soda tests just described. The specimens were allowed to thaw and soak in water during the day, and were hung up and frozen at night. The experiments lasted the same number of periods, two of which were of 36 hours, to insure thorough freezing. Through the courtesy of Mr. Fellowes Morgan, I was able to use the cold storage room under the piers of the Brooklyn Bridge. The temperature of the cold room was


Curves of the action of Na_2SO_4 and Frost on Building Stones.
The Ordinates are proportional to the loss in weight in parts in 10,000.
Ordinate (a) = ¼ scale.
Ordinate (b) = ⅛ scale.
b
a
Na_2SO_4
Frost
(1)
(2)
(3)
(4)
(5)
(6)
(7)
(8)
(9)
(51)
(11)
(22)
(10)
(12)
Coarse Dolomite
Medium Dolomite
Limestone
Coarse Granite
Medium Granite
Fine Granite
Gneiss
Norite
Decomposed Sandstone
"
Sandstone
"
Fine Sandstone
Brick

from 4° to 10° Fahr. (—19° to —23° Cent.), and the specimens were left to thaw and soak during the day in a room with the temperature at about 85° Fahr. (30° Cent.).

After the completion of the tests the specimens were allowed to soak in water for as long as it took to remove all traces of sulphate of soda in the other series of experiments, after which they were carefully dried and weighed. During the progress of these experiments the deterioration was so slight that the effect was scarcely noticeable, the sandstones only showing the effect by a slight residue in the bottom of the pails in which the experiments were made. The results are given in tabular form.

RESULTS OF EXPERIMENTS WITH SULPHATE OF SODA.

No.	SPECIMENS TESTED.	Original weight in grams.	Loss in weight in grams.	Loss in weight in parts in 10 000.
1..	Coarse crystalline dolomitic marble	71.9020	0.0775	10.78
2..	Medium crystalline dolomitic marble	93.8861	0.1597	17.01
3..	Fine-grained limestone	67.0964	0.1744	25.99
4..	Coarse-grained granite	71.8648	0.1115	15.51
5..	Medium-grained red granite	56.4939	0.0370	6.55
6..	Fine-grained gray granite	43.5910	0.0225	5.16
7..	Rather fine grained gneiss	61.8687	0.0392	6.33
8..	Norite, "Au Sable" granite	35.1173	0.0135	3.84
9..	Decomposed sandstone	39.4294	1.9010	482.12
10..	Very fine-grained sandstone	37.7760	0.1800	47.65
11..	Sandstone	28.0325	0.4070	145.18
12..	Pressed brick	37.4025	0.0930	24.86
51..	Decomposed sandstone	22.9660	3.7235	1 621.31
52..	Sandstone	24.9001	0.1381	57.78

RESULTS OF EXPERIMENTS WITH FROST.

No.	SPECIMENS TESTED.	Original weight in grams.	Loss of weight in grams.	Loss in weight in parts in 10 000.
1..	Coarse crystalline dolomitic marble	63.6407	0.0197	3.10
2..	Medium crystalline dolomitic marble	93.9851	0.0216	2.30
3..	Fine-grained limestone	55.4518	0.0115	2.07
4..	Coarse-grained granite	52.2787	0.0072	1.38
5..	Medium-grained red granite	63.4693	0.0112	1.76
6..	Fine-grained gray granite	58.6149	Very slight, about same as No. 5.	
7..	Rather fine-grained gneiss	52.7260	Very slight, about same as No. 5.	
8..	Norite, "Au Sable" granite	44.4665	Very slight, less than No. 5.	
9..	Decomposed sandstone	38.4055	0.2640	68.74
10..	Very fine-grained sandstone	39.5120	0.0420	10.63
11..	Sandstone	21.9437	0.0312	14.21
12..	Pressed brick	37.1790	0.0255	6.86
51..	Decomposed sandstone	24.1020	0.0610	25.31
52..	Sandstone	20.2285	0.0180	8.89

Attention is called to the following observations, resulting from a study of the curves on page 242. Among the limestones the effect of sulphate of soda seems to have been strongest on the pure limestone (No. 3), while the action of frost seems to have been most marked on the very coarse-grained dolomite (No. 1). The granites seem to have been affected by both methods in the ratio of the extent of the decomposition of the feldspar, and the size of the grains.

The sandstones were all much more affected, by both processes, than either the limestones or granites. The decomposed specimens (Nos. 9 and 51) were very much disintegrated by the sulphate of soda; while the very fine-grained specimen (No. 10) was the least affected by both methods.

The specimen of brick (No. 12) seems to have been more acted on by both methods than either the limestones or granites. Of all the specimens tested the compact Norite ("Au Sable" granite, No. 8) showed the least disintegration.

These results and those of former experiments, although not carried on under the same conditions for accurate comparison, all seem to prove beyond a doubt that the action of sulphate of soda is much more powerful than that of freezing water.

An interesting problem has been suggested by the experiments just completed. Is there any definite relation in time between the effects of frost and sulphate of soda? That is, would the effect of sulphate of soda, for a certain period of time, be equivalent to the effect of frost extending over a definite longer period of time?

Detailed Description of Specimens of Building Stones Tested.

No. 1. Dolomitic marble, very coarse crystalline, from Pleasantville, Westchester County, N. Y. It is called "Snow Flake Marble," and is much harder than the Vermont marbles (Smock). It was perfectly fresh, and was very tough, splitting with difficulty.

No. 2. Dolomitic marble, white medium crystalline, from Tuckahoe, N. Y. According to Smock it seems to be durable, is harder than the Vermont and foreign marbles, and freezing and thawing produce no apparent effect upon it. In preparing the specimens it split quite easily. Little scales of light-colored mica were observed to be present.

No. 3. Limestone, white and fine grained. It is almost pure calcium carbonate, dissolving very rapidly in cold dilute hydrochloric acid, and showing only a trace of magnesium. In preparing the specimens it split very easily.

No. 4. Granite, gray and rather coarse grained, with large feldspar crystals, from Gallager's Quarry, Me. A section showed that the feldspar was chiefly decomposed orthoclase, very little plagioclase being noticed. Biotite, more or less decomposed, was the important mica, and a few little crystals of apatite were observed. During the process of cutting and grinding the section, it was noticed to be softer than the red Jonesborough granite (No. 5). It split rather easily, the feldspar being quite decomposed.

No. 5. Granite, red and medium grained from Jonesborough, Washington County, Me. A section showed the feldspar to consist principally of very much decomposed pink and cream-colored orthoclase. Very little plagioclase was noticed. A little green mica was present, and the quartz contained many pores. In preparing the section, it was noticed to be hard, cutting and grinding with difficulty.

No. 6. Granite, gray and fine grained from Hallowell, Kennebec County, Me. It split quite easily in preparing the specimens, although rather hard. A section showed the very fine-grained character of the rock. The feldspar was very little decomposed. Orthoclase, microcline and plagioclase were noticed. The acicular crystals, mentioned by Smock as occurring in this granite, were observed. Most of the mica was quite fresh, a little green chloritic matter, however, was seen. This granite works easily on account of the small proportion of quartz and the presence of mica.

No. 7. Gneiss, rather fine grained and stratified, from Bedford, N. Y. It split quite easily in preparing the specimens. A section showed the presence of very light-colored mica shreds, giving a stratified appearance to the section. The quartz was in small, rounded grains. Orthoclase, more or less decomposed, a very little plagioclase and microcline, and a few grains of hornblende were present. The rock as a whole, appeared fresh, and there seemed to be a good "bond" between the grains.

No. 8. Norite, called the "Au Sable" granite, from Keeseville, N. Y. It is hard and tough, takes a beautiful polish, and splits with difficulty. According to Smock, it stands high temperature well. A

section showed that it was composed of fresh plagioclase crystals, with a very perfect "bond" and close contact, and pyroxene. In the two sections examined most of the pyroxene was the orthorhombic pyroxene, hypersthene, although the monoclinic pyroxene was also present.

No. 9. Sandstone (brownstone) decomposed, taken from an old building, pulled down June, 1893, in Smith Street, near Schermerhorn Street, Brooklyn.

No. 10. Sandstone (brownstone) fresh, very fine grain, and showing distinct stratification, from Belleville, N. J. According to Smock, the cementing material is chiefly iron and lime, and the stone is unchanged by freezing and thawing. Numerous little scales of white mica were noticed in this sandstone, and it effervesced in hydrochloric acid. A section showed the sandstone to be composed chiefly of very small, sharp fragments of quartz. Little grains of calcite were also noticed.

No. 11. Sandstone (brownstone), fresh, rather more granular in structure than No. 10, and showing no stratification, from Belleville, N. J. According to Smock this stone is not affected by freezing and thawing, and the cementing material is chiefly iron and lime. The sandstone effervesced in hydrochloric acid, and a section showed it to be composed principally of rather large angular fragments of quartz. A few grains of decomposed feldspar and calcite appeared to be present, also.

No. 12. Brick, sample taken from a good specimen of pressed brick.

No. 51 and 52 were pieces of Nos. 9 and 11 that had been soaked in dilute hydrochloric acid for a week.

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DISCUSSION.

L. DeCoppet Berg, M. Am. Soc. C. E., called attention to the small amount of practical information published regarding most important subjects, such, for instance, as the relative corrosion of cast iron, wrought iron and steel, and referred particularly to the almost entire lack of knowledge concerning the weathering qualities of stone. Mr. Berg.

Speaking of another paper recently sent to him by the author, he took exception to a statement contained therein, that coarse-grained stone withstood the action of frost better than fine grained, his experience apparently proving the contrary; and he took occasion to ask the author whether the experiments under discussion did not prove the reverse, viz., that the finer grained stone is safer in resisting the weather.

As an architect, he stated that it was customary with his profession to select a stone on account of its beautiful color or because it harmonizes with its surroundings, or on account of a desired finish which it will readily take—first, of course, making as sure as possible, by the opinions of experts, that the stone will weather well. When the

Mr. Berg. stones are built in, every precaution is taken to get them of even quality, on their natural bed, etc., and yet after one winter's exposure, frequently, certain stones will begin to show discolorations, and, as year after year goes by, will deteriorate more and more, until, after seven or eight years, they are almost entirely disintegrated, although most of the neighboring stones are perhaps still perfect.

He therefore wished to ask the author if there was any simple practical test, which could be applied at the building, to find out whether two stones of apparently the same quality were actually so.

He added that the only method of preserving stone known in this city is a process with which all are familiar, that of heating the stone to such a degree that it will melt and absorb paraffine applied to the surface to a depth of a quarter of an inch or more, thus filling the pores with a water-resisting surface. The objections to this method are, that it seems to bring out black marks or discolorations in the stone, presumably due to the carbonization of vegetable matter contained in the pores, and that in the course of time the process wears off and has to be renewed, as was done in the case of the obelisk in Central Park.

He concluded by saying that it was still questionable, therefore, whether the repeated heatings of the stone did not deteriorate it more in the long run than the frost would have done had the surface been left untreated.

Mr. Owen. JAMES OWEN, M. Am. Soc. C. E., said that he was not particularly well informed on the possibly intricate question of testing stone by either the soda process or by the artificial frost process, but had had considerable experience for a number of years with different varieties of sandstones and more particularly with the sandstones from New Jersey alluded to. Some years ago Professor Egleston enunciated the idea of the optical examination of stones for building purposes and other uses, and in that statement he outlined a definition of a sandstone for building purposes and drew a broad line between the two. He stated that as a rule, in almost all cases, a sandstone with a siliceous binding was a permanent structural material, and that sandstone with an argillaceous binding was liable to deterioration by frost.

He then remarked that in his experience in the use of the New Jersey sandstones he had found out that it was perfectly practicable to select a stone which can be considered absolutely weather proof, and stated that the coarser the stone, the better it was.

The particular element, he continued, in this question of weathering of sandstones brings in their inherent structure. Sandstone might be defined as beginning almost a pure alumina, then the combinations of alumina and silica, and then working up to a pure silica. In the proper line between the alumina and the silica is where the question of judgment enters. In that line also comes the structure of the stone itself. If it is aluminous in its binding, the question of deterioration enters.

Mr. Owen. The weathering of shale stones when laid on their bed is practically imperceptible, but that same stone, if laid on its wrong bed, will begin to spawl off very markedly in a very short period. The dressing of stone had also an effect; if stone be well rubbed, it is more permanent than one rough faced or with a projecting arras. A coarse-grained silicious-bound sandstone can usually be obtained from any quarry if it is insisted on. He condemned quarries not containing this kind of stone.

The peculiar formation of sandstone ridges in New Jersey, he observed, is that one range of hills will have good hard sandstone almost up to the surface of the ground; the next range will consist of what are called there "callous" stone, which is a clay sandstone, so that by experience it is possible to judge on opening a quarry the character of the stone in the one adjacent to it.

He asked if, in the weight test, weight of the particles falling from the stone added to the stone itself gave the same weight as the original specimen? Also, if all the crystals of soda were washed out of the stone?

The question of absolute cohesion of the stone occasionally enters as a factor of permanence. He then cited an instance in which quite a number of the stones from a good quarry checked and the fractures followed no line and no law: they seemed to start off at one point and zigzag across in any direction; there appeared to be insufficient cohesion to stand weathering, although otherwise a good stone.

Mr. Croes. J. J. R. CROES, M. Am. Soc. C. E., inquired how thin or how thick the transparent sections of stone spoken of by the author in the verbal discussion are; how they are prepared, and the method of examination.

CORRESPONDENCE.

Mr. Gerber. E. GERBER, M. Am. Soc. C. E., observed that in 1890, during the construction of the new bridge across the Mississippi at Winona, Minn., the question of suitable stone for the piers arose, and as a limestone was to be found in the immediate vicinity, it was desirable to know its weathering qualities as compared with other stones whose qualities were known. Accordingly he was directed by the Consulting Engineer, George S. Morison, M. Am. Soc. C. E., to make tests by the soda process on stone from Bedford, Ind.; Stone City, Ia.; Mankato and Winona, Minn. The process as conducted by others, he added, was investigated in such records as could be readily obtained, and the method afterwards pursued determined upon as the one which would most nearly approximate the action of freezing and thawing.

Mr. Gerber. He then described the stones from the localities above mentioned, as follows: Bedford stone is an oolitic limestone composed of 97% of pure carbonate of lime; it is of very uniform grain, either gray or bluish in color; has no definite lines of stratification, and is comparatively soft. The Stone City stone, commonly known as Anamosa stone, is a true dolomite, containing 57% carbonate of lime and 41% carbonate of magnesia; it is light buff in color, more or less porous, and shows stratification very plainly. Its grain is not uniform, varying from fine to coarse; it is somewhat softer than the Bedford stone. The Mankato stone, suitable for bridge work, is almost a true dolomite, containing 52% carbonate of lime, 36% carbonate of magnesia and 7% sulphate of lime. It is of a yellowish color, generally fine grained, but has occasional sand holes and is about as hard as Bedford stone. The Winona stone is a magnesian limestone containing 47% of carbonate of lime, 21% carbonate of magnesia, 4% sulphate of lime, and 24% of insoluble matter, mostly silica. It is light buff in color, very fine grained except for the presence of numerous sand holes, and is generally quite hard.

Two samples of each kind of stone, he stated, were taken, making eight in all. The samples were cut approximately to 2-in. cubes, and were then carefully ground to true faces and generally to sharp corners. This treatment was of some value in observing the action of the sulphate of soda on the different parts of the stone. After preparing in the manner above described, the stones were dried in air for about a week and then subjected for about 8 hours to a hot-air bath at 212° Fahr., after which they were all carefully weighed. The samples were then all immersed in water for 24 hours, to determine the relative amount of absorption. After this they were again carefully weighed and then dried for 3 days, the object being to remove as much water as possible before saturating with the sulphate of sodium solution.

A cold saturated solution of sulphate of sodium was then prepared and the stones immersed in this solution and left for 24 hours. At the end of 24 hours the stones were removed from the solution and hung up in the atmosphere at a temperature of about 70° for about 24 hours, to allow the sodium sulphate to crystallize. At the end of the 24 hours the samples were carefully washed with a wash bottle and the washings caught in a beaker, it being at that time proposed to weigh the material detached from the cubes. At the first washing the detached particles of stone were plainly visible in every case; in some, however, the quantity was considerably greater than in others, as the table below indicates. The quantity of detached material from the first washing being very small in some cases, and various other difficulties presenting themselves, the plan of weighing the washings was abandoned and they were not preserved. It was noticed, however,

throughout the experiment, that the washings after each crystallization, in the case of each sample, plainly showed detached material. After the first washing, the samples were again immersed in sodium sulphate, this time for 4 hours, and the drying repeated as before. In this manner the samples were immersed in the solution in all eight times, one time for 24 hours, and seven times for 4 hours each. The periods of drying varied from 20 to 72 hours, according to the condition of the atmosphere. All samples were treated exactly alike; that is, they were all immersed at the same instant and taken out of the baths at the same instant, this being accomplished by having the beakers in which they were immersed standing in a row and the samples all attached to one wooden stick by fine wires. Mr. Gerber.

After the last crystallization the samples were soaked in clean water for a week, the water being frequently changed to wash out the sodium sulphate. After soaking, the samples were dried in ordinary atmosphere for about a week and then subjected to the hot bath the same as before the first weighing, the intention being to have the drying after the experiment as nearly as possible the same as the drying before the first weighing. The table on page 252 gives the results of the experiments.

He added that the faces of the different samples all being ground very smooth, the action of the sulphate of sodium on the different kinds of stone was readily observed. The Bedford stone was acted on very uniformly throughout the entire surface, the resulting faces being something like a sand paper made up of fine pebbles; the corners remained comparatively sharp, and no more violent action was observed on them than on the faces. The Stone City stone was also acted on pretty uniformly, but not as much so as the Bedford, and the corners were broken to a considerable extent. The Mankato stone was affected more in the sand holes than on the smooth portions, and the same was true of the Winona stone; in both cases the harder portions were comparatively smooth after the test, and in the latter case scarcely any action was noticeable on the hard, smooth portions of the faces.

Immediately after the above experiment had been completed, the results obtained being sufficient for the purpose in hand, he had learned, through the efforts of Mr. John Warder, M. E., that he could subject these samples to actual freezing and thawing in a cold storage warehouse. The Union Cold Storage Company courteously consented to have the experiment continued in their warehouse. The samples used above, together with three others, were, therefore, taken to this warehouse, and subjected to alternate freezing and thawing, in the following manner: The stones were immersed during the day in water, and at night hung up in the cold storage rooms, which were at an average temperature of from 0 to 4° Fahr. This part of the work was done by one of the employees of the warehouse, and the process

TABLE OF RESULTS OF SODIUM SULPHATE TESTS.

Sample marked.	From.	Description.	Weight dry. Grams.	Weight after being immersed in water 24 hours. Grams.	Amount of water absorbed. Grams.	Percentage of original weight absorbed.	Weight after sodium crystallizations and drying. Grams.	Amount lost. Grams.	Percentage Loss.
I.....	Bedford	Specimens homogeneous and as nearly alike as may be. Faces	311.12	324.02	12.90	4.15	309.98	1.14	00.37
II.....	"	true and smooth, corners not very sharp.	310.38	322.57	12.19	3.93	309.27	1.11	00.36
III.....	Stone City	Fine grained, faces true and smooth, corners sharp. Stratification lines visible, but faint.	278.99	304.81	25.82	9.25	277.20	1.69	00.61
IV.....	"	More porous, faces and corners as above. Stratification lines very plain.	300.51	317.00	16.49	5.45	299.76	.75	00.25
V.....	Mankato	Sand holes in both, faces indented, corners generally	338.31	350.41	12.10	3.58	337.54	.77	00.23
VI.....	"	sharp. Hard parts smooth.	330.40	342.23	11.93	3.61	329.77	.63	00.19
VII.....	Winona	Full of sand holes, faces indented, corners battered more	344.73	350.11	5.38	1.56	344.35	.38	00.11
VIII.....	"	or less. Hard parts smooth.	343.39	348.37	4.98	1.45	342.95	.44	00.13

was continued for about 10 days or two weeks, no exact record of the number of exposures being kept, when the samples were again dried, as in the previous experiment, and weighed. The results of this test are given in the following table : Mr. Gerber.

Sample marked.	From	Weight before freezing. Grams.	Weight after freezing and drying. Grams.	Loss. Grams.	Loss, per-centage.	Percentage of loss by freezing as compared with soda test.
I.....	Bedford.....	309.98	309.68	.30	00.097	26.3
II.....	"	309.27	308.95	.32	00.103	29.8
III.....	Stone City.....	277.20	276.83	.37	00.134	21.9
IV.....	"	299.76	299.60	.16	00.053	21.3
V.....	Mankato	337.54	337.16	.38	00.113	49.4
VI.....	"	329.77	329.42	.35	00.106	55.5
VII.....	Winona.....	344.35	344.18	.17	00.049	44.7
VIII.....	"	342.95	342.80	.15	00.043	34.1
IX.....	Ashland, brown-stone.....	261.57	261.40	.17	00.068	
X.....	Ashland, brown-stone.....	294.19	293.93	.26	00.088	
XI.....	Hannibal, Mo....	331.56	331.05	.51	00.154	

He then remarked that the time of the one experiment in this case is not the same as the time of the other, and a comparison is, therefore, not as valuable as in the case of the paper under discussion. It is of interest, however, to compare the loss during the actual freezing process with the loss by the sulphate of sodium test, and this comparison is given in the last column of the above table. The relative effect of freezing and the action of sulphate of sodium is evidently not the same for different kinds of stone. Within the narrow limits of the experiments, however, it appears to be reasonably constant for the same kind of stone.

He observed, further, that close-grained stones are less likely to be acted on by frost than coarser ones. The same is true, also, of the author's experiment. Omitting Nos. 5, 6, 7, 8 and 51 in the author's experiments, for obvious reasons, the average loss by freezing is 16.5% of that by the soda test.

In the experiment which had been conducted under his direction this average is 35.4 per cent. When it is considered that the freezing in the latter case was more frequent than the sodium crystallization, being possibly double the number of times these averages approach each other ; and while the experiments described are too few in number and on too great a variety of stones for the small total number of experiments, it is perhaps not unreasonable to expect that a comparatively definite relation in time may yet be determined between the effects of frost and of sulphate of soda. That sulphate of soda is a much more powerful agent than frost in the disintegration of stone he believed to be verified by his experiments.

Mr. Phillips. HIRAM PHILLIPS, Assoc. M. Am. Soc. C. E., wrote that while Director of the Physical Laboratory of the Missouri State University in 1890–92, he had made a large number of tests of building stones and brick with sulphate of soda. The records were burned and the results of the tests are therefore given from memory, but he believed them to be accurate deductions.

All specimens were first prepared by grinding into 2-in. cubes (no chiseling nor hammering being allowed). They were then put in a drier with a temperature of 130° Fahr. and left there 24 hours. The specific gravity was then determined, after which they were soaked in water of 60° Fahr. for 24 hours, to find their absorption. For each series of tests five specimens were prepared and the mean of the five taken as a final result. The parallel faces were made exactly true by a thin coat of plaster of paris, and they were crushed between steel cushions in an Olson testing machine:

1. A set of specimens were crushed after having been placed in a drier 24 hours, and a mean of their results taken as a basis of comparison.

2. A set was subjected to the soda test by being boiled in a solution 30 minutes, and then suspended in a dark room 23 hours; this was repeated 30 times.

3. A set was subjected to the soda test by being boiled in a strong solution 3 hours, and then placed in a dark room 21 hours; this was repeated 30 times.

4. A set was subjected to the soda test by being boiled 6 hours, and then placed in a dark room for 18 hours, which was repeated 30 times.

5. A set was also boiled 12 hours in soda solution, and placed in a dark room 12 hours; this was repeated 30 times.

6. A set was boiled in pure water 12 hours, and placed in drier 12 hours; this was repeated 30 times.

7. A set was subjected to boiling in soda solution 12 hours and then left to stand 12 hours in the solution; this was repeated 30 times.

8. A set was placed in a retort, steamed slightly, the air exhausted, and the soda solution at 100° Fahr. forced into the retort under 20 lbs. pressure and left 30 minutes, and the set finally placed in a dark room 24 hours. This was also repeated 30 times.

The results of these tests led him to draw the following conclusions:

First.—Rocks of low specific gravity and high absorption were almost uniformly affected by boiling in the solution, the effects being more strongly marked the longer the boiling was continued and the stronger the solution used; it was greatest on sandstones and porous brick.

Second.—The results obtained by test No. 7 showed that the specimens were disintegrated almost as fast as in test No. 5, which would

seem to indicate that it was the boiling in the solution that caused the disintegration and not the forming of crystals. Mr. Phillips.

Third.—In test No. 6 the rocks were seldom affected, and when injury occurred it seemed to be in developing cracks and seams which had escaped detection in the preparation of the specimens.

Fourth.—The results from test No. 8 showed less injury than from any other test which would seem to indicate that more injury results from boiling than from the soda crystals.

Fifth.—There was less injury to the specimens in test No. 6 than in test No. 5.

He explained that as the greater number of these specimens were from rock of known durability, the results seemed to indicate that the action of the sulphate of soda is much more severe than actual freezing. A considerable number were entirely disintegrated in tests Nos. 5 and 7, some of them being from sandstone curbing that had stood for 30 years without appreciable decay.

He concluded by pointing out that while some good building stone will not withstand the sulphate of soda test, an engineer would be justified in looking with suspicion on any stone that rapidly disintegrates under the process, especially if accompanied by low specific gravity and high absorptive powers.

F. Collingwood, M. Am. Soc. C. E., remarked that some years ago he tested several samples of sandstones as to their durability. They were much larger than those experimented upon by the author, being about 4-in. cubes. The method pursued was to dry and then weigh them, soak them in water for a day, expose them to a temperature of about 0° Fahr. for a day, and finally dry for another day over the brick work of a steam boiler. This process was repeated for about a month with occasional weighings; but the results were discrepant, and indicated that a longer period of drying was required. The experiments were repeated for another month with two-day periods of drying. The final results were not very satisfactory, however, and seemed to show that a much longer period would be required by this method to obtain positive indications as to the value of a building stone. Mr. Collingwood.

The use of smaller specimens, he continued, is no doubt desirable, provided care be taken that they fairly represent the stone to be tested. It would seem, also, that the method pursued by the author promises to be much more useful than the freezing process.

Lea McI. Luquer, Ph. D., answering Mr. Berg, stated that it is generally accepted as true that, "other things being equal, the value of a stone for building purposes is inversely as its porosity or absorbing power," but that G. P. Merrill, in his report on building and ornamental stones, says that this rule may have exceptions. He called attention to the fact that a coarsely porous stone will not only take up a large proportion of water, but will also part with it readily; and in Mr. Luquer.